



US Army Corps
of Engineers

AD-A219 763

DTIC FILE COPY

DREDGING OPERATIONS TECHNICAL
SUPPORT PROGRAM

2

TECHNICAL REPORT 5-90-1

METHODS OF DETERMINING THE LONG-TERM FATE OF DREDGED MATERIAL FOR AQUATIC DISPOSAL SITES

Mark S. Dwyer, Technical Editor
Environmental Laboratory

James J. Hayes, Contributor
Sediment Transport Research Center

Joseph P. Lister, William H. McAnally, Jr., Contributors
Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
8409 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

DTIC
ELECTE
MAR 28 1990
S DCS D



February 1990
Final Report

Approved for Release by NSA on 09-11-2013 pursuant to E.O. 13526



DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE				Form Approved OMB No. 0704-0188	
1a. REPORT SECURITY CLASSIFICATION Unclassified			1b. RESTRICTIVE MARKINGS		
2a. SECURITY CLASSIFICATION AUTHORITY			3. DISTRIBUTION/AVAILABILITY OF REPORT		
2b. DECLASSIFICATION/DOWNGRADING SCHEDULE			Approved for public release; distribution unlimited.		
4. PERFORMING ORGANIZATION REPORT NUMBER(S) Technical Report D-90-1			5. MONITORING ORGANIZATION REPORT NUMBER(S)		
6a. NAME OF PERFORMING ORGANIZATION See reverse.		6b. OFFICE SYMBOL (If applicable)	7a. NAME OF MONITORING ORGANIZATION		
6c. ADDRESS (City, State, and ZIP Code) 3909 Halls Ferry Road Vicksburg, MS 39180-6199			7b. ADDRESS (City, State, and ZIP Code)		
8a. NAME OF FUNDING/SPONSORING ORGANIZATION US Army Corps of Engineers		8b. OFFICE SYMBOL (If applicable)	9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER		
8c. ADDRESS (City, State, and ZIP Code) Washington, DC 20314-1000			10. SOURCE OF FUNDING NUMBERS		
			PROGRAM ELEMENT NO	PROJECT NO	TASK NO
					WORK UNIT ACCESSION NO.
11. TITLE (Include Security Classification) Methods of Determining the Long-Term Fate of Dredged Material for Aquatic Disposal Sites					
12. PERSONAL AUTHOR(S) See reverse.					
13a. TYPE OF REPORT Final report		13b. TIME COVERED FROM _____ TO _____		14. DATE OF REPORT (Year, Month, Day) February 1990	
15. PAGE COUNT 205					
16. SUPPLEMENTARY NOTATION Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.					
17. COSATI CODES			18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)		
FIELD	GROUP	SUB-GROUP	See reverse.		
19. ABSTRACT (Continue on reverse if necessary and identify by block number)					
<p>To manage an open-water dredged material disposal site, it is essential to know the physical capacity of the site (i.e., how much material should be dumped at the site and what the capability is of the material to remain onsite under various environmental conditions of waves and currents). Long-term management of aquatic disposal sites also requires an understanding of how much area the disposal mound encompasses, when the mound encroaches on the site boundaries, how much material leaves the site, and perhaps where the material ultimately goes.</p> <p>The purpose of this report is to identify methods that can be used to develop information concerning the long-term fate of dredged material disposed at aquatic sites. The methods are broken into two major categories: (a) methods of analysis for mound resuspension and dynamics and (b) methods of analysis for transport and redeposition of mound</p> <p>(Continued)</p>					
20. DISTRIBUTION/AVAILABILITY OF ABSTRACT <input checked="" type="checkbox"/> UNCLASSIFIED/UNLIMITED <input type="checkbox"/> SAME AS RPT <input type="checkbox"/> DTIC USERS			21. ABSTRACT SECURITY CLASSIFICATION Unclassified		
22a. NAME OF RESPONSIBLE INDIVIDUAL			22b. TELEPHONE (Include Area Code)		22c. OFFICE SYMBOL

6a. PERFORMING ORGANIZATION (Continued).

USAEWES, Environmental Laboratory,
Coastal Engineering Research Center, and
Hydraulics Laboratory

12. PERSONAL AUTHORS (Continued).

Dortch, Mark S. (Technical Editor); Hales, Lyndell Z.; Letter, Joseph V.;
McAnally, William H., Jr. (Contributors)

18. SUBJECT TERMS (Continued).

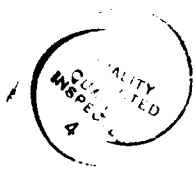
Aquatic disposal sites	Sediment resuspension
Disposal mound	Sediment transport
Dredged material	Site capacity
Long-term fate	

19. ABSTRACT (Continued).

material. For each of these two categories, four basic approaches are reviewed:
(a) steady-state analytical methods; (b) time- and rate-dependent analytical methods;
(c) physical and numerical modeling; and (d) measurements through field and laboratory
studies.

Other sections of the report are devoted to discussions of physical processes and
study recommendations. Additional details of the methods of analysis are provided in four
appendixes.

Accession For	
NTIS CRA&I	<input checked="" type="checkbox"/>
DTIC TAB	<input type="checkbox"/>
Unannounced	<input type="checkbox"/>
Justification	
By	
Dist	
Availability Codes	
Dist	Availability Codes
A-1	



PREFACE

This study was sponsored by the Dredging Operations Technical Support (DOTS) Program, which is funded by the Headquarters, US Army Corps of Engineers (HQUSACE), through the Dredging Division. The DOTS is managed through the Environmental Effects of Dredging Programs (EEDP) of the Environmental Laboratory (EL), US Army Engineer Waterways Experiment Station (WES). Dr. Robert M. Engler was EEDP Manager, and Mr. Thomas R. Patin was DOTS Coordinator in EEDP. The work was monitored by Mr. David B. Mathis, Dredging Division, HQUSACE.

The principal investigators for this study were Dr. Lyndell Z. Hales, Coastal Processes Branch, Research Division, Coastal Engineering Research Center (CERC), and Mr. Joseph V. Letter, Estuarine Simulation Branch, Estuaries Division (ED), Hydraulics Laboratory (HL). Technical coordination of the study was provided by Mr. Mark S. Dortch, Chief, Water Quality Modeling Group, Ecosystem Research and Simulation Division (ERSD), EL. The study was conducted under the general supervision of Mr. Donald L. Robey, Chief, ERSD, and Dr. John Harrison, Chief, EL; Dr. James R. Houston, Chief, CERC; and Mr. Frank A. Herrmann, Jr., Chief, HL.

Contributing authors for this report were Dr. Hales, Mr. William H. McAnally, Jr., Chief, ED, Mr. Dortch, and Mr. Letter. Technical editing of this report was provided by Mr. Dortch. Technical review was provided by Mr. E. C. McNair of the Estuarine Engineering Branch, ED. The report was edited for publication by Ms. Jessica S. Ruff and Ms. Jamie W. Leach of the WES Information Technology Division.

Commander and Director of WES was COL Larry B. Fulton, EN. Technical Director was Dr. Robert W. Whalin.

The report should be cited as follows:

Dortch, M. S., tech. ed. 1990. "Methods of Determining the Long-Term Fate of Dredged Material for Aquatic Disposal Sites," Technical Report D-90-1, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

CONTENTS

	<u>Page</u>
PREFACE	1
CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT	4
PART I: INTRODUCTION	5
Background	5
Purpose and Scope	5
PART II: PROCESS DESCRIPTIONS	8
Short-Term Fate and Mound Formation	8
Resuspension of Mound Material	19
Transport and Redeposition of Mound Material	24
PART III: METHODS OF ANALYSIS FOR MOUND RESUSPENSION AND DYNAMICS	29
Requirements for Resuspension	29
Steady-State Analytical Methods	49
Time- and Rate-Dependent Analytical Methods	61
Modeling	68
Field and Laboratory Studies	76
PART IV: METHODS OF ANALYSIS FOR TRANSPORT AND REDEPOSITION OF MOUND MATERIAL	88
Steady-State Analytical Methods	88
Time- and Rate-Dependent Analytical Methods	95
Modeling	100
Field and Laboratory Studies	109
PART V: RECOMMENDATIONS	110
PART VI: SUMMARY	113
Purpose of the Study	113
Process Descriptions	114
Steady-State Analytical Methods	128
Time- and Rate-Dependent Analytical Methods	135
Modeling	138
Field and Laboratory Studies	143
Recommendations	148
REFERENCES	151
APPENDIX A: FIELD AND LABORATORY TESTS	A1
Field Tests	A2
Laboratory Tests	A8
APPENDIX B: PHYSICAL MODELING	B1
Use of Physical Models	B1
Advantages and Disadvantage of Physical Models	B4
Field Data Requirements	B5
Verification	B11
Time and Cost Estimates	B15

	<u>Page</u>
APPENDIX C: NUMERICAL MODELS	C1
Uses of Numerical Models	C1
Available Numerical Models	C2
Advantages and Disadvantage of Numerical Models	C7
Field Data Requirements	C9
Time and Cost Estimates	C12
APPENDIX D: HYBRID MODELS	D1

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
inches	2.54	centimetres
knots (international)	0.5144444	metres per second
miles (US nautical)	1.852	kilometres
miles (US statute)	1.609347	kilometres
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.4535924	kilograms
square feet	0.09290304	square metres

METHODS OF DETERMINING THE LONG-TERM FATE OF DREDGED MATERIAL
FOR AQUATIC DISPOSAL SITES

PART I: INTRODUCTION

Background

1. The US Army Corps of Engineers (USACE) and the US Environmental Protection Agency (EPA) have been directed by Section 103 of the Marine Protection, Research, and Sanctuaries Act (MPRSA) of 1972, Public Law (PL) 92-532, to cooperatively evaluate the impact of dredged material disposal into ocean waters. The Corps has the responsibility of regulating and managing the open ocean disposal sites. The primary intent of Section 103 of PL 92-532 is to regulate and strictly limit adverse ecological effects of ocean dumping.

2. In order to manage an open-water dredged material disposal site, it is essential to know the physical capacity of the site: (a) how much material should be dumped in a particular location; and (b) what the capability is of the material to remain onsite under various environmental conditions of waves and currents.

3. Once dredged material has been placed in open water and has settled to the bottom, it becomes susceptible to erosion, resuspension, and transport away from the disposal area by bottom currents. These currents may be induced by various sources, including tidal currents, wind-wave action, density flows, riverine discharges, or storm events. Understanding processes responsible for resuspension and transport is critical to site management since it, in part, determines the storage capacity of a disposal site. Long-term management of disposal sites requires an understanding of how much area the disposal mound encompasses, when the mound encroaches on the site boundaries, how much material leaves the site, and perhaps where the material ultimately goes.

Purpose and Scope

4. The purpose of this report is to identify methods that can be used to develop information concerning the long-term fate of dredged material disposed at aquatic sites. This information will be useful to those involved

with the long-term management of dredged material aquatic disposal sites (DMADS). The various quantitative techniques presented will help answer such physical questions as:

- a. How much material should be disposed at a site and for how long?
- b. What should be the rate and timing of disposal?
- c. How much material is lost from the mound over time and when will the loss rate be unacceptable?
- d. When will the mound depth be limiting?
- e. When will the mound reach the site boundary?
- f. Where is resuspended material redeposited?

5. A method of analysis that would address any one of the first five questions above should provide information about the other four. To address the first five questions, the method of analysis must determine such things as:

- a. Mounding characteristics due to disposal operations.
- b. The conditions that cause resuspension of mound material.
- c. The erosion and transport rate of mound material.
- d. Armoring and changes in sedimentology of the mound.

6. Methods of analysis that will allow one to address the sixth question (paragraph 4f) are different from the first five because transport away from the site and redeposition must be considered. Therefore, the methods presented herein are broken into two major categories: (a) methods of analysis for mound resuspension and dynamics and (b) methods of analysis for transport and redeposition of mound material.

7. A variety of analysis methods could be used for each of the major categories above, ranging from simple desk-top analytical methods to sophisticated numerical models. Four approaches are presented for both of the two major categories. These approaches are:

- a. Steady-state analytical methods.
- b. Time- and rate-dependent analytical methods.
- c. Modeling (physical, numerical, and hybrid).
- d. Measurements through field and laboratory studies.

8. The first approach is the easiest to implement, but it yields the least information. This approach assumes steady or constant conditions and would be representative of long-term average conditions. Such an analysis could fail to show the extreme results that can occur during episodic events.

The second approach requires more effort to apply than the first but includes the effects of extreme events by taking into account exceedance frequency of ambient conditions and variations in rate-dependent processes. The third approach includes various numerical and physical modeling activities. Modeling can offer the most detail, but it can also require significant effort. Finally, field monitoring of erosion and deposition and laboratory studies, such as flume studies of erosional characteristics, may be useful.

9. The types of aquatic sites considered herein are primarily open-water sites, including oceans, Great Lakes, and estuaries or tidal waterways. The methods presented were based on existing techniques and do not include any new development.

10. In addition to the discussions on methods of analysis (Parts III and IV), attention is given to descriptions of physical processes related to DMADS, such as material placement, resuspension, and transport and redeposition (Part II) and to future research and development needs and recommendations (Part V). Due to the size of this report, the summary (Part VI) has been written to stand alone if necessary. Thus, the reader can obtain from the summary the more important information provided in the report. As a result, the summary is longer and more detailed than usual. Appendixes A-D provide additional details of the analysis methods.

PART II: PROCESS DESCRIPTIONS

11. The fate of dredged material disposed in DMADS can be discussed in terms of short term and long term. The short-term fate has to do with the dredged material placement, mound formation, and the near field (in the vicinity of the DMADS) and short term (within minutes and hours following disposal dispersion). The long-term fate has to do with erosion and resuspension of mound material over longer time frames, such as years, and the redeposition of this material. This report deals with the long-term fate of DMADS. Although consolidation of mound material could be considered a long-term process and certainly affects the mound erosion and resuspension processes, consolidation is not discussed in this report. Consolidation is the subject of another report (Poindexter, in preparation) that was part of the overall project that funded this work.

12. Part II discusses the processes that affect (a) the short-term fate of dredged material and the mound formation, (b) the resuspension of mound material, and (c) the transport/redeposition of mound material. Although resuspension and transport/redeposition of mound material are the focus of this report, mound formation requires some discussion because it is the starting point for the long-term processes. Methods for determining the short-term fate and mound formation are presented in a report by Johnson (in preparation), which was also part of the overall project that funded this work.

Short-Term Fate and Mound Formation

13. In assessing the capacity of a proposed or existing DMADS, it is important to have a conceptual understanding of the manner in which dredged material behaves when dumped. As discussed by Bokuniewicz et al. (1978), the movement of material from a hopper dredge or barge to an at-rest position on the bottom can be partitioned into two basic transport phases: (a) convective descent and (b) impact or collapse on the bottom with the formation of a horizontal base surge. Release to the receiving water is the only aspect of dredged material placement over which direct control can be exercised by conventional dredge operations. Once the material is released from the dredge, the mechanics of the two basic transport phases is essentially beyond manipulation by operators. (It is theoretically possible to control to a degree

the horizontal base surge by some type of underwater containment, but this may be impractical.) A schematic representation of the processes at work as dredged material is released from a hopper dredge to an aquatic disposal site is shown in Figure 1.

Convective descent

14. Field data have been obtained by Bokuniewicz et al. (1978) during the actual release of dredged material at three aquatic disposal sites: the New York Bight; Ashtabula, Ohio; and Rochester, N. Y. Among these study sites, the water depths ranged from 15 to 67 m, and currents in the receiving water ranged from 0 to 0.7 m/sec. A wide range of sea states and weather conditions were encountered during the studies. The dredged material being placed ranged from highly fluid riverine mud to dense marine silt, and the quantities discharged in any single operation ranged from 380 to 6,120 cu m. Despite this wide range of conditions, it was found that the same basic sequence of placement processes took place at each locality. In general, the dredged material was transported to the bottom in a narrowly defined jet of high-density fluid and as blocks or clods of cohesive soil. Upon impact with the bottom, a radially expanding surge was formed that dispersed dredged material away from the impact area. This material was deposited in the form of a ring having a radius of several hundred metres.

15. Regarding the phase known as convective descent, for the situation where the dredged material is assumed to be dumped essentially instantaneously (such as from a hopper dredge or a dump barge), the dumped material possesses an initial downward momentum and a density greater than that of the surrounding water. This results in forces that cause the material to settle in the form of a cloud, or density current, rather than as individual particles. As the cloud settles, shear stresses develop at the interface between the moving cloud and the ambient water, resulting in dissipation of the initial momentum and the creation of turbulent eddies that entrain ambient fluid. In the case of clouds possessing an initial momentum, vortex rings form at the time of release and tend to cause deeper penetration of the ambient water. The material that falls as clods acquires terminal speed after falling through a small fraction of the water depth and then descends to the bottom at a nearly constant speed. Any distribution of material between jet and clod descent is possible; the proportion of material in the two forms has a major effect on the structure of the resultant deposit at the disposal site.

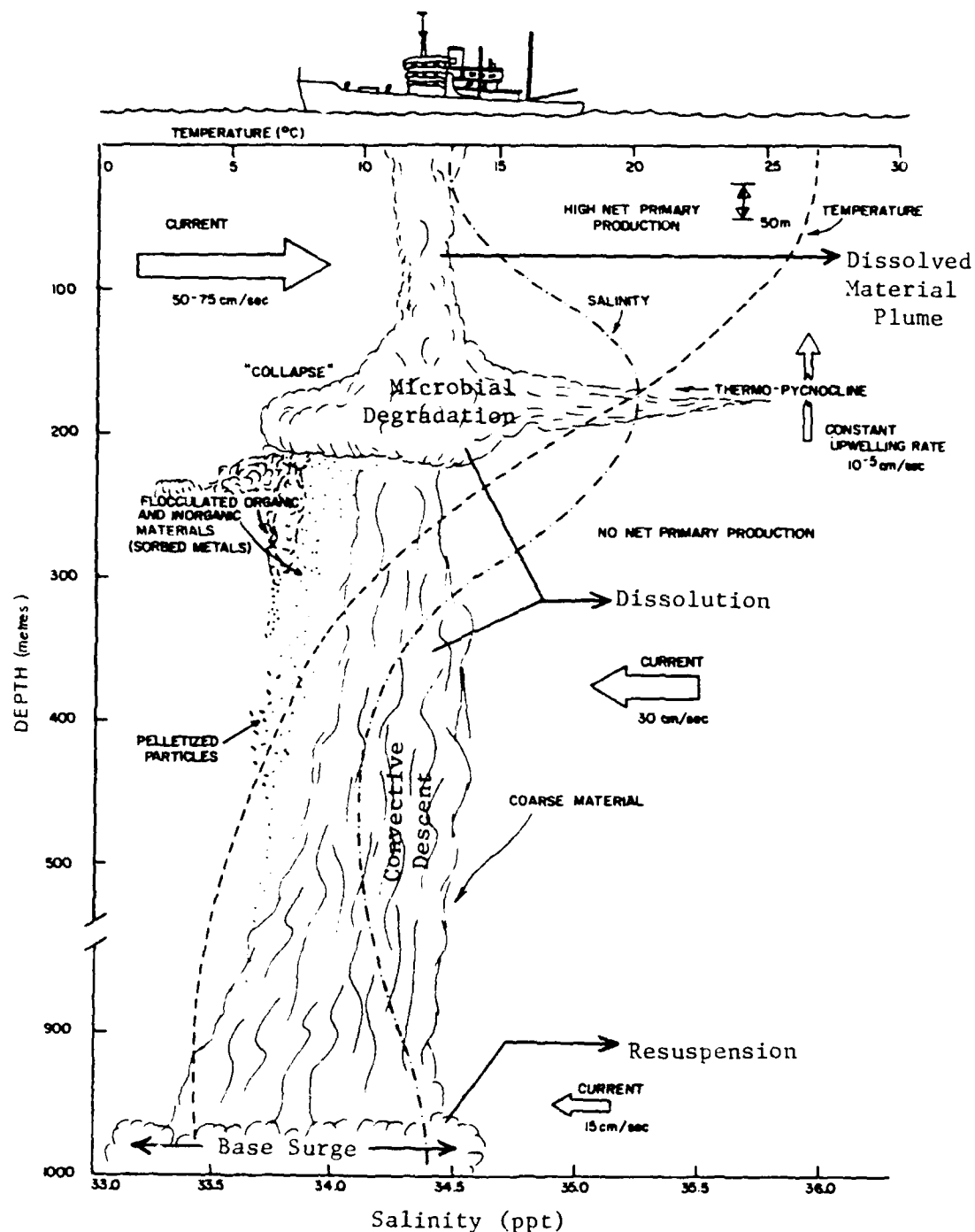


Figure 1. Schematic representation of dredged material placement at a two-layered, aquatic, deep-water site with a strong thermocline. The figure shows the depth of high net primary production (25 to 50 m), spreading of fine material on the thermocline, and minimal effect of upwelling as material passes through the thermocline (after Pequegnat 1978)

16. The jet was observed by Bokuniewicz et al. (1978) to fall at a nearly constant speed and to entrain a large volume of ambient water during transit from the surface to the bottom. For example, the volume of fluid reaching the bottom in the jet may be 70 times the volume released at the surface. Because of the large entrainment and the corresponding reduction in jet density, the jet quickly attains the lateral speed of any current flowing in the receiving water. Its impact point can be predicted with good accuracy if the current is known. The descent of the jet sets up circulation patterns in the ambient water inward toward the discharge point on the surface and outward on the bottom. The resultant inflow around the hull of the dredge or scow helps contain the dredged material in a narrowly defined zone of descent. The speed of this convective descent was measured by Bokuniewicz et al. (1978) with precision scientific instrumentation and was found consistently to be about 1 m/sec.

17. Instantaneous dumping of dredged material in relatively shallow water produces a rapid convective descent of the material with a vertical velocity on the order of 1 m/sec. Settling velocities calculated for individual particles do not apply during this form of transport. Since the time during which the cloud is in contact with the upper portions of the water column is a minute or less, ambient water currents (except near the bottom) are of little consequence except as they affect the transport of any turbidity cloud that may be generated during the descent. If near-bottom currents are low, precision dumping may proceed under almost any current condition occurring in the upper portions of the water column, except for turbidity cloud considerations (Johanson et al. 1976).

Horizontal base surge

18. The second phase of vertical transport occurs when the cloud begins a dynamic collapse on the bottom, characterized by horizontal spreading. Collapse is driven primarily by a pressure force and is resisted by inertial and frictional forces. Dynamic collapse will occur when the cloud encounters a boundary, either a density layer (pycnocline) or the bottom. In the case of precision dumping of dredged material into a specific site, it is important that the cloud penetrate through any density layer and reach the bottom. In general, sudden releases of fairly large quantities of dredged material in shallow water will penetrate a density layer and impact on the bottom. The cloud will flatten out and appear somewhat like a disk as it assumes a

horizontal circular shape (assuming a flat bottom and no obstructions) with a small vertical dimension. Under these conditions, flow will continue in the form of a density or turbidity current.

19. If a clod of dredged material impacts the bottom at high speed, it will disintegrate, and the contained material will be dispersed. If the impact speed is low, the clod will be deposited intact. The condition necessary to avoid disintegration is that the kinetic energy of the clod at arrival be dissipated in plastic deformation of the clod or the bottom before material failure occurs. Since the kinetic energy per unit mass of a falling clod increases as the clod size increases, it is expected that there is an upper bound to the size of clods that can be deposited on the bottom intact. This carries implications regarding precision dumping.

20. Turbidity, or density, flows of sediments released from dredging operations have also been reported by May (1973). He reported on aquatic disposal from channel and shell dredging by hydraulic dredges, where almost all the sediment settled very rapidly and was transported along the bottom as a separate, flocculated density layer. The sediment that was not deposited immediately under the dredge was transported in the density flow or base surge. Concentrations of 10,000 mg/l were found within 400 ft* of the discharge point, and concentrations over 1,000 mg/l extended out at least 1,800 ft.

21. At Ashtabula and Rochester, the base surge spread radially outward in the shape of a thin expanding toroid of turbid water. Both its thickness and speed decreased as its radius increased. As the surge proceeded outward, it shed behind a thin, slowly moving cloud of suspended dredged material that settled to the lake floor. The entrainment of ambient water and friction eventually caused the velocity of the surge to decrease to the point where all its contained sediment was deposited. The initial energy of the surge and the rate of energy dissipation determine the range of the base surge, the area of the bottom that will be covered by dredged material, and the form and thickness of this deposit. Ideally, the deposition of dredged material is expected to occur in a ring around the impact point.

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 4.

22. To adequately describe a bottom surge, it is necessary to know its velocity as a function of distance from the impact point, its thickness, and the concentration of solids contained therein. If sufficient data are available, it is possible to determine whether erosion or deposition occurs at a given radial distance, whether additional ambient water is entrained, and how rapidly kinetic energy is lost. These data may then be used to estimate the size of the deposit that will be formed on any given aquatic disposal site.

23. Figure 2 is an example of a travel-time diagram constructed by Bokuniewicz et al. (1978) from data taken at Ashtabula. The time is figured from the time of hopper door opening, which was signaled from the dredge. If the travel-time curve is to represent only the run of the surge over the bottom, both time and distance should be figured from the impact of dredged material on the bottom. A time correction can be calculated from the water depth and the descent speed, and a distance correction from the descent speed and the ambient current. An additional correction may be required for the displacement of the surge itself by the current in the overlying water. If there were no friction between the surge and the water above, application of this correction would be inappropriate, but if this friction is larger than that of the surge over the bottom, it should be applied. All three corrections were applied to the travel-time data of Figure 2. The descent corrections make only a small change in the travel-time curve, but the corrections for background current produce a wide dispersion of the data points. Thus, it appears that the advance of the bottom (base) surge is independent of the flow of the ambient water.

24. The thickness of the base surge was found to depend on water depth: the greater the depth, the thicker the surge. The best data were obtained at the Great Lakes sites by Bokuniewicz et al. (1978). As the water depth at the disposal site increased from 20 to 50 m, the greatest thickness of the surge increased from 4 to 7 m. This result was expected since, at the greater depth, the volume of water entrained during descent was greater but the speed of the surge over the bottom was not changed appreciably. The surge thickness was also observed to be relatively large at the New York Bight site. While the water depth was greater there, the quantity of dredged material released was also much greater, and the surge spreading speed was higher. There were insufficient data to separate the effects of all of these variables in determining surge thickness. Figure 3 is a contour diagram defining the thickness

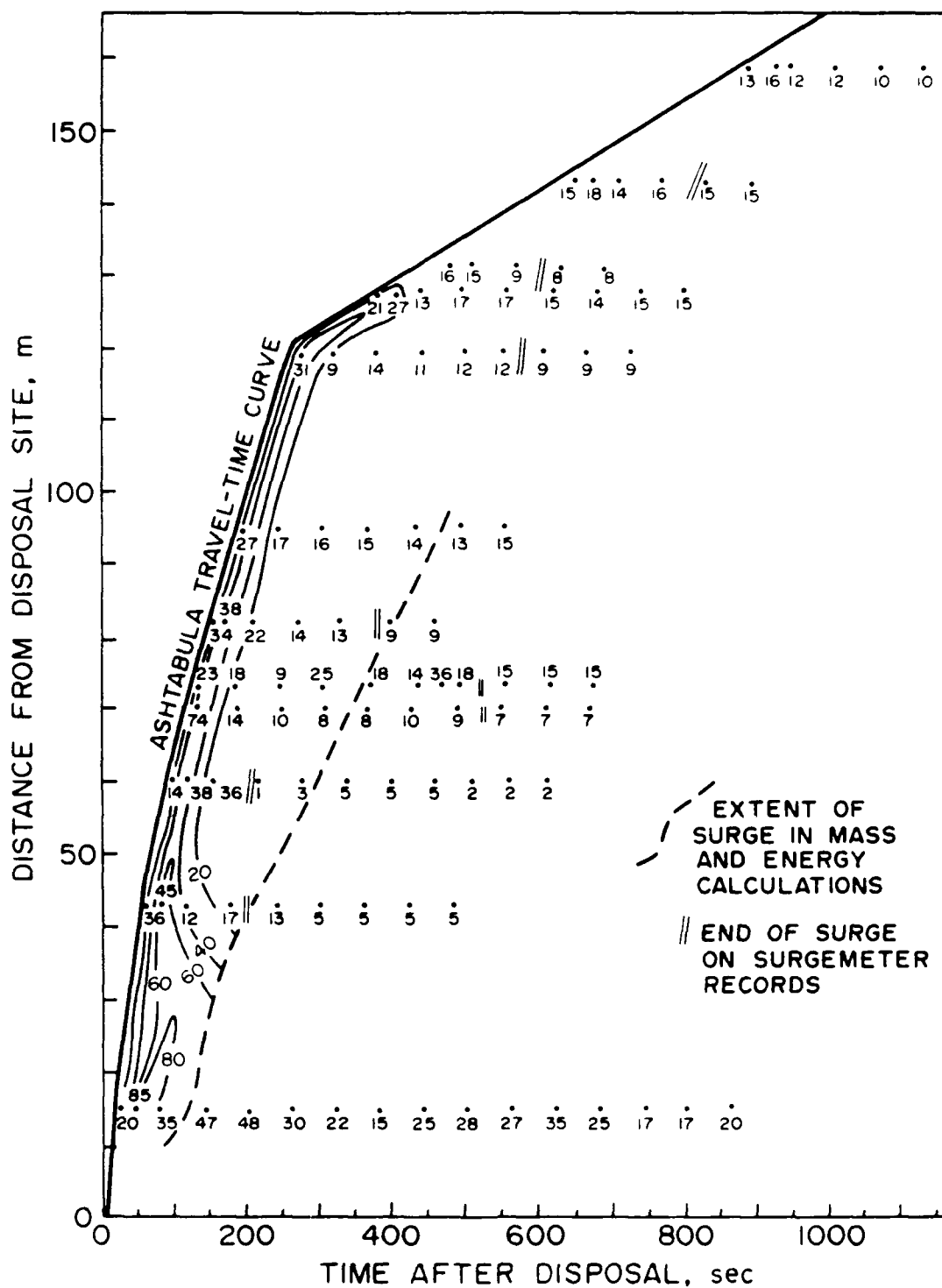


Figure 2. Contour diagram defining velocity of base surge after passage of head of surge. Data adjusted to the Ashtabula travel-time curve. Contour interval = 20 cm/sec (after Bokuniewicz et al. 1978)

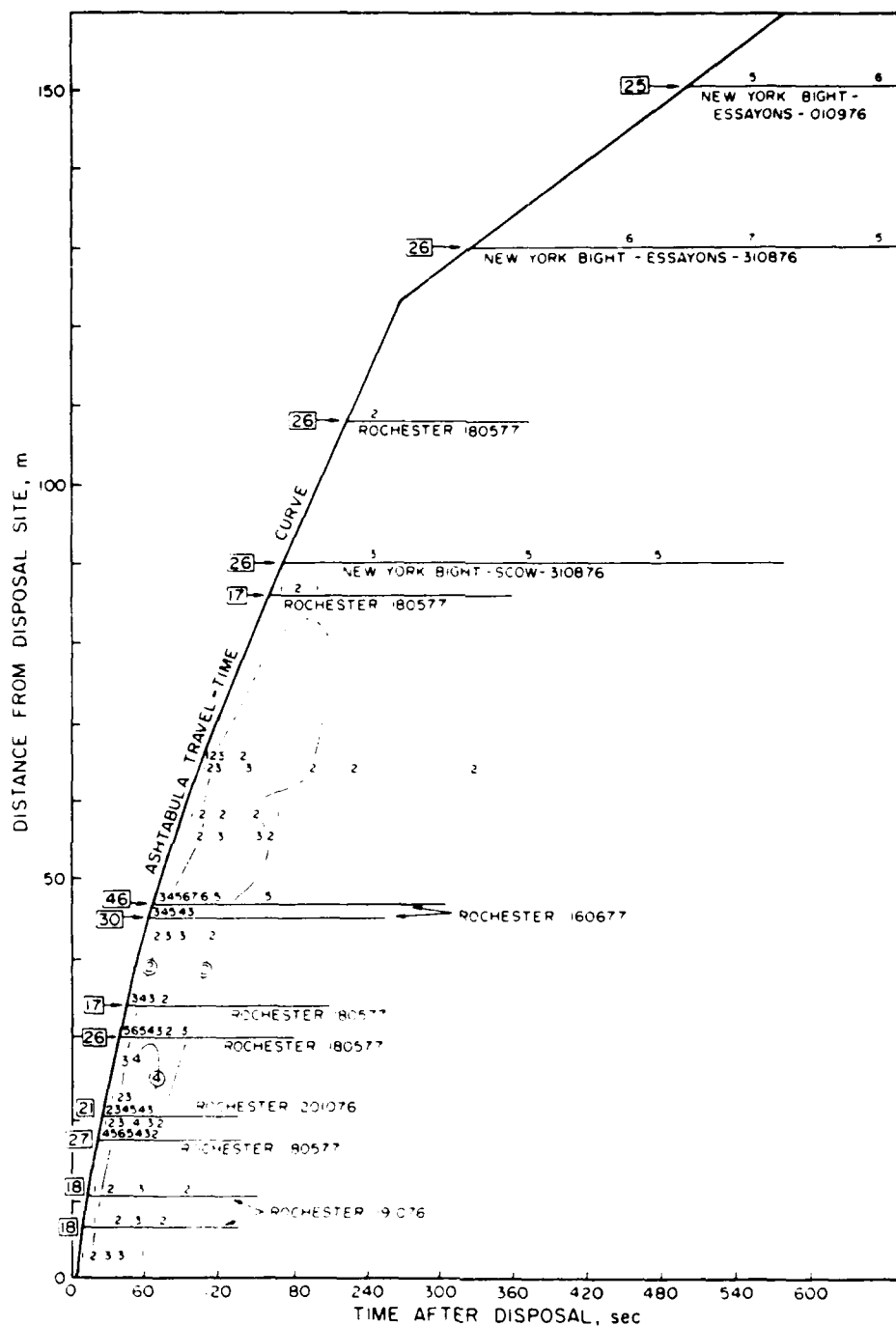


Figure 3. Contour diagram defining height of base surge. Numbers in boxes are depth in metres. Data adjusted to Ashtabula travel-time curve. Contour interval = 2 m (after Bokuniewicz et al. 1978)

of the base surge for the Ashtabula, Rochester, and New York Bight data after adjusting to the Ashtabula travel-time curve.

25. The concentration of solids suspended in the base surge was determined from pumped water samples and from transmissometers. At the Great Lakes sites, concentrations were as high as 11 g/l within about 50 m of the impact point. Three minutes after the head of the surge had passed, the concentrations were down to about 1 g/l, and returned to background values in less than 15 min. These data are displayed in Figure 4.

Mound formation

26. The vast majority of the disposed dredged material rapidly descends to the bottom of the disposal area where it accumulates under the discharge point in the form of a low-gradient fluid mud mound overlying the existing bottom sediment (Figure 5). If the discharge point of a hydraulic pipeline dredge is moved as the dredge advances, a series of mounds will develop. The majority of the mounded material is usually high-density (nonflowing) fluid mud that is covered by a surface layer of low-density (flowing or nonflowing) fluid mud. Under quiescent conditions, more than 98 percent of the sediment in the mudflow remains in the fluid mud layer at concentrations greater than 10 ppt, while the remaining 2 percent may be resuspended by mixing with the overlying water at the fluid mud surface. Fluid mud will tend to flow downhill as long as the bottom slope is approximately 1 percent or greater.

27. If bottom slopes are not great enough to maintain mudflows, the fluid mud will stop and begin to consolidate. When suspended sediment concentrations exceed 200 ppt, the fluid mud can no longer flow freely but will accumulate around the discharge point in a low-gradient (e.g., 1:500) fluid mud mound (Barnard 1978). At the water column/fluid mud interface, the solids concentration increases very abruptly from perhaps a few tenths of a part per thousand in the water to approximately 200 ppt in the fluid mud. The solids concentration within the fluid mud increases above 200 ppt at a slower rate with depth until it reaches normal sediment densities. Thicker layers of fluid mud reach their final degree of consolidation more rapidly than thinner ones. Depending on the thickness of the fluid mud and its sedimentation/concentration characteristics, complete consolidation of a fluid mud mound may require from one to several years. In those situations where material dredged by bucket or clamshell is of slurry consistency, the above description is generally applicable. More commonly, however, muddy sediment dredged by a

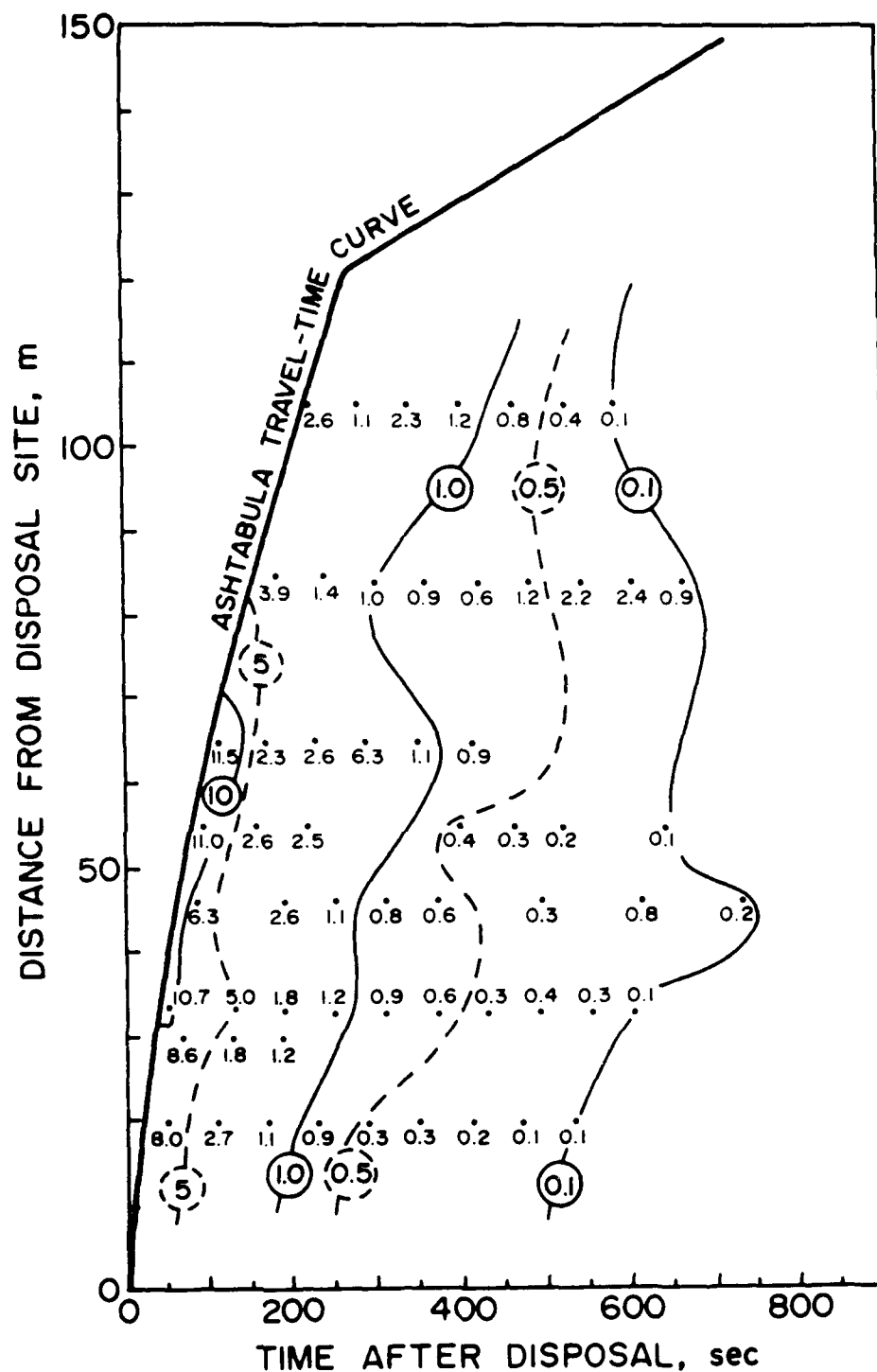


Figure 4. Contour diagram defining the concentration of suspended material in the base surge. Data from pumped sample concentration in grams per litre, adjusted to Ashtabula travel-time curve. Contour interval as noted (after Bokuniewicz et al. 1978)

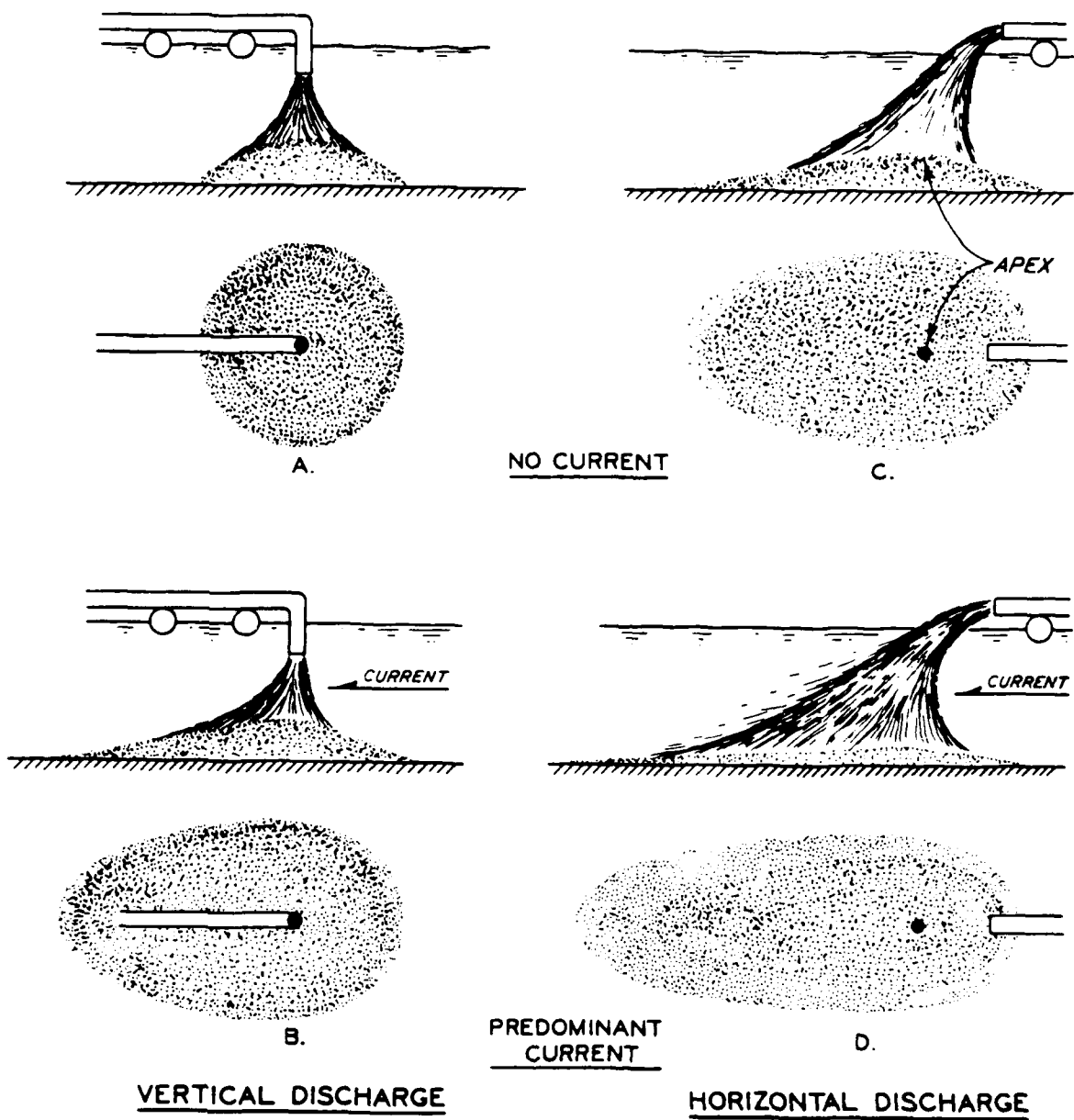


Figure 5. Effect of discharge angle and predominant current direction on the shape of a fluid mud mound (after USACE 1983)

clamshell remains in large clumps and descends to the bottom in this form. The clumps may break apart somewhat on impact, but such material tends to accumulate in irregular mounds under the discharge vessel rather than move outward from the discharge point. Whatever the dredging method, sandy sediment tends to mound directly beneath the discharge pipe or vessel.

Resuspension of Mound Material

28. When water begins to flow over a bed of loose particles, hydrodynamic forces are exerted upon the particles; as the flow intensity increases, the magnitude of the hydrodynamic forces on the particles increases. A condition is eventually reached where the particles are unable to resist the forces and movement is initiated. Disposal mound dynamics depend on forces, mound characteristics, armoring, and other processes such as biological activity.

Forces causing resuspension

29. Dominant forces contributing to erosion and transport are stresses by currents caused by tides, density gradients, waves, winds, and episodic events such as storms. Sediment erosion and transport are also influenced by the nature of the sediments themselves--their size, physicochemical, and consolidation properties--and characteristics of the mound. Other factors such as armoring and biological activity also play roles in resuspension of mound material.

30. Tides. The astronomical forces of the moon and sun cause tides in the ocean that have both vertical and horizontal motions. These tidal motions, combined with topographic features, give rise to a rotary type of tidal current in the open ocean and along the sea coast. This current varies with locality, depending upon the character of the tide, the water depth, and the configuration of the coast. In any locality, however, these tidal currents repeat themselves as regularly as the tides to which they are related. In the open ocean and wide estuaries, the tidal currents usually are rotating due to the effect of the Coriolis force; i.e., from hour to hour, the currents change in both direction and magnitude. In narrow estuaries, tidal currents tend to be bidirectional--ebb and flood.

31. In estuarine systems where the tidal prism (volume of water entering from the sea during flood phase of the tide) is large in relation to the

daily freshwater runoff, currents are oscillatory with pronounced ebb and flood phases. Examples include the Hudson River estuary and the Chesapeake Bay. In a few estuaries, notably that of the Mississippi River, riverflow dominates tidal currents with the result that the flow is usually uniformly downstream, showing the tidal effect only in speed and stage variation. Broad shallow estuaries with small river inflow and small tides, such as Mobile Bay, are subject to dominant wind-induced unidirectional currents. Depending on estuarine geometry, reversing tidal currents may be strictly bidirectional, with a predominant ebb direction and flood direction, and a slack period of near-zero current speed, or they may be rotary, swinging through a range of compass headings during the ebb and flood phases with or without a slack period. Tidal information, including predictions of water levels, tidal ranges, and local datum planes for the coasts of North and South America, is published by the US National Ocean Survey.

32. Waves. The ability of water waves to transport bottom sediment is related to the magnitude of the shear stress exerted by the wave motion on the bed and dynamic pressure changes under the waves. Oscillatory fluid motion associated with surface gravity waves exerts shear stresses on the bottom that are often several times larger than shear stresses caused by unidirectional currents of the same magnitude because of the pressure fluctuations. Thus, the importance of wave motion in initiating and transporting sediments in a coastal environment is apparent, as the stresses produced by wave motion may put sediments into suspension where they can be transported by currents of a magnitude insufficient to initiate sediment motion.

33. Density gradients. Density gradients caused by differences in dissolved solids content and water temperature impose both vertical and horizontal circulation patterns on the tidal currents. The pattern of stronger upstream currents near the bottom and stronger downstream currents near the surface can lead to net upstream flow in the lower layers of the water column. The pattern can be expressed as a flow predominance (Simmons 1966) that indicates the percentage of flow in either direction at a given point in the water column. Between the downstream predominance of the river and the upstream predominance of the bottom currents in the lower estuary, null areas will occur in which there is no net flow predominance in either direction. This, of course, has serious implications for transport of sediments and contaminants through the estuary.

34. Wind. Other significant aspects of estuarine hydrodynamics include wind-induced currents and seiching produced by wind stress. Wind-induced currents have a complex structure that includes generally downwind surface currents and upwind bottom currents in deep channels. These currents may alter sediment transport patterns or, in some cases, may be the primary transport mechanism.

35. Episodic events. The passage of a meteorological event such as a storm or hurricane results in the generation of extreme waves on the ocean surface and, for a semienclosed region such as an estuary or lake, a change in water level and sometimes in currents. These events may result in disturbance of the sediment on the bottom. When dredged material placed at an aquatic disposal site decreases the local depth substantially, its susceptibility to material resuspension and dispersion by storms is increased. Consequently, an upper bound on the amount of material that can be accommodated at any given site (the site capacity) can be set in terms of the degree of dispersion that is acceptable. In order to be able to describe the importance of storms as a source of disturbance of the bottom, it is necessary to have a measure of the frequency of occurrence of storms of different magnitudes affecting the site.

Disposal-mound characteristics

36. Data on the physical properties of existing disposal mounds are extremely meager (Basco et al. 1974). A considerable amount of information has been collected regarding the chemistry of the material, presumably due to environmental concerns, but these data have very little bearing on the long-term fate of the mound. The physical properties that have been reported are those that were obtained from highly disturbed samples. Undisturbed samples will ultimately yield the most information of use in evaluating the long-term fate of an aquatic disposal mound.

37. Composition. The fate of a dredged material disposal mound is almost entirely a function of the material being deposited, because of the minor influx of materials from outside sources. The material in the mound will differ from the material at the dredging site, primarily by the amount of fines that has been carried away during deposition. The composition of a disposal mound is also strongly influenced by whether the disposal material comes from a new area project or from dredging of existing channels. Material from new channel dredging will commonly be clay that becomes armored with fine silt or shell. During deposition, these artificially formed mud balls, along with

the coarser particles, will settle first, bringing about an easily observable interface between the natural bottom at the disposal site and the disposal mound. Mounds from maintenance dredging, conversely, are often observed to have a more homogeneous consistency.

38. When consideration is being given to site capacity of nondispersive dredged material disposed in aquatic sites, one condition that should be met is that the site should be located where there is natural deposition or non-disturbance of sediment of a grain size equal to or less than the grain size of the dredged material. With reference to the Long Island Sound, deposition of silt occurs in the central and western basins. The New Haven disposal site is located in a region where silt is accumulating at a rate of about 8 g/sq m/year. Eleven other designated disposal sites in Long Island Sound are also located in depositional environments, although none are located where the most rapid sedimentation occurs.

39. Configuration. The topography of an aquatic dredged material disposal mound depends to a great extent on the original topography of the area prior to any placement. If the dredged material is deposited on a sloping bottom, gravitational forces will cause material to move downslope and will tend to spread the material over larger areas, much as a turbidity current or density flow is known to exist in the ocean's continental slope region. However, if the material is placed between ridges or in a depression zone, the spreading will be inhibited and the overall effect will be to smooth the troughs and peaks of the existing undulating bed.

40. The ultimate long-term topography of the mound depends to a great extent on the type of disposal operation that was employed. Whether the final bathymetry will form a ridge, cone, or some other formation depends on the precision with which the disposal operation is performed. Some disposal may take place prior to reaching the disposal site, or at locations not precisely designated for dumping, because of imprecise navigation techniques or the desire to minimize dump haul distance. For whatever reason, unless the material is placed as prescribed, the resulting bathymetry of the disposal mound will be significantly affected.

41. The characteristics of the dredged material and the existing currents of the area affect the mound topography. It was found by Briggs (1970) that disposal in the Upper Chesapeake Bay resulted in material covering an area approximately five times that of the disposal area originally selected.

The characteristics of the material were such that the maximum slope measured about 1:100 (average slopes were 1:500). Bathymetric surveys at different stages of disposal operations in the Rhode Island Sound by Saila et al. (1972) showed the maximum slopes of disposal sites to be about 9:100. If free dumping occurs, the slopes of the mounds should be indicative of the shear strength of the materials involved. However, the effect of currents is probably just as important in arriving at the ultimate slope of the aquatic disposal mound.

42. Consolidation. Consolidation is a decrease in thickness of a saturated layer due to the decrease in the void ratio under the action of an effective overburden pressure. The degree of consolidation is important because it can be related to shear strength characteristics of the dredged material disposal mound. Salem and Krizek (1973) determined that dredged material was slightly more compressible than typical inorganic soils, and they noted that, at low intensities of loading, the void ratio of some samples actually increased with time rather than decreased. This was attributed to the generation of gases in the material, which tended to counteract the applied load and allowed expansion of the sample. In general, freshly deposited dredged material at an aquatic disposal site will be in a highly underconsolidated state. With the passage of time, excess water will escape from the voids and, upon completion of this process, the material will be normally consolidated. If some of the overlying material is subsequently eroded after the mound has become normally consolidated, the remaining material will be in an overconsolidated state as some of the overburden pressure has been removed. Under normal conditions, different portions of a disposal mound may exist in underconsolidation, normal consolidation, and overconsolidation.

Disposal site armoring

43. Extensive laboratory flume tests have been conducted on the sorting and armoring effects for various time intervals of flow. It has been concluded that when a bed of graded particles (some of which are too large to be moved by the available tractive forces) is allowed to erode, the nonmoving particles accumulate at the bed surface, reducing the area of the bed from which particles can scour. The reduction in the rate of transport has been found to be proportional to the accumulation of nonmoving particles on the bed surface. This accumulation on the bed surface causes an increase in the effective roughness and also increases the limiting grain size for

nonmovement. Harrison (1950) found that for a layer of nonmoving particles, the thickness of one particle was effective in preventing scour in rivers. An extension of this reasoning can be advanced to aquatic dredged material disposal mounds where the analogy exists. Because of turbulent currents and unsteady pressure gradients in the wave field, it is not anticipated that a layer of uniform-sized grains will accumulate at the mound surface; however, a definite coarsening or armoring capability will occur as long as finer particles are being placed in resuspension.

Biological activity

44. According to Basco et al. (1974), activity of some burrowing organisms may increase erosion of mounds of dredged material. In contrast, the activity of tube-building animals within the mounds may slow down the rate of erosion. Such animals can cover mounds with dense mats of soft tubes that may protect the dredged material from erosion and even act as traps for fine particles. Thus, the effects of biological activity remain uncertain and uncontrollable.

Transport and Redeposition of Mound Material

45. It is known that resuspension and redeposition of movable material (cohesive or noncohesive) is functionally related to the magnitudes of physical stresses (forces) that induce such movement. If the natural environmental forces existing in the region are less than those forces required for particle motion, the particle will remain at rest. If, however, the available stresses are greater than those required to place material in motion, particle motion will be initiated. Transport will continue until the moving particles enter a water mass where the hydrodynamic forces decrease to a value less than that necessary for maintaining transport, at which time the particles will settle from the water column. To perform the appropriate analysis of a DMADS regarding long-term site capacity, it is necessary to know the magnitudes of the physical forces that exist in the region. If this information is unavailable, a suitable, comprehensive field data-collection program should be initiated to obtain the necessary data.

Transport

46. Sediment transport consists of three physical processes: sediment particles are entrained into the water column; sediment particles are

transported by the motions of the water column; and sediment particles deposit or redeposit on the bed. While the particles are entrained by the water column, motion may exist as either bedload or suspended load. Bedload is defined as the sediment that moves on or in frequent contact with the bottom. Suspended load is that material transported within the water column, maintained above the bottom by the turbulence in the water column. Usually particles which enter the suspended load stage are quickly dispersed, whereas particles in the bedload stage travel in spurts for short distances. Complex entrainment-transport-deposition cycles lead to large spatial and temporal variations in sediment distribution within any given estuarine, coastal, or lake environment.

47. The ultimate destination (fate) of suspended sediment particles depends on the particle size and character and the magnitude of the currents and associated turbulence that keeps the particles in suspension. The smaller silt and clay particles have extremely slow settling velocities (less than 0.001 mm/sec) and will remain suspended (because of their size and shape) until aggregation into composite particles permits settling.

48. The mode of transport is dependent upon size, shape, and submerged weight of sediment particles and the character of flow. Coarse-grained, non-cohesive sands (greater than 62 μ) are transported as individual grains, tending to be frequently in contact with the bed. Fine-grained sediment (less than 62 μ) travels mostly in suspension, approaching the bed only when flow intensity is very low or when, in the case of cohesive sediment, individual grains collect into composite particles that settle through the water column. The transport processes of coarse and fine sediments are substantially different.

49. Coarse sediment particles are transported as individual grains. A grain on the bed begins to move when lift and drag exerted on it by the flow overcome the restraining forces of grain weight and friction with adjacent grains. Once in motion, a grain may roll or hop a short distance or be entrained in the flow. During live bed transport, there is a continuous interchange of grains between the bed and the sediment in transport. Deposition and erosion of grains occur simultaneously. When the flow's capacity to transport sediment at a point is equal to the sediment supply from upstream, deposition and erosion are in balance and a stable bed results. Any deficiency in sediment supply or to the flow's transporting capability causes

degradation or aggradation of the bed. Coarse, noncohesive sediment often tends to move in undulating bed forms in which individual grains move only short distances as the bed form migrates.

50. Individual clay particles have exceedingly small settling velocities and usually will not settle under their own weight, even in still water, because thermal motion of water molecules will keep the particles in suspension. Only when clay particles aggregate, forming porous composite particles of a large number of individual particles, can settling begin. Surface electrical charges on the particles attract a layer of ions, making them mutually repulsive except at very short distances and preventing significant aggregation. If two particles collide in spite of this repulsion, short-range attractive forces bind them together into aggregates. The strength of this adhesion depends on interparticle distances. This aggregation process, called flocculation, increases with increasing number of particle collisions caused by higher sediment concentration, by increased turbulence in the flow, and by the increased presence of dissolved salts whose ions suppress electrical repulsion between particles. At some upper limit, turbulence may hinder flocculation by breaking aggregates as rapidly as they are formed. Aggregates grow larger by colliding with individual particles and other aggregates when differing settling velocities and flow velocity gradients permit them to be captured by faster moving ones. Other aggregation processes include agglomeration by filter-feeding organisms and chemical cementation.

51. The interchange between a cohesive bed and a suspension with depositable sediment is a function of the bed shear stress. Below a certain minimum shear, depositable sediment will always deposit; above the minimum and up to some maximum shear, available sediment will deposit at a rate dependent on bed shear; above the maximum shear, sediment will not deposit and erosion may occur at a rate dependent upon bed shear. The critical shear stress for erosion of a sediment layer may increase as consolidation changes the sediment structure, forming more particle bonds between aggregates.

52. Alterations of either sediment supply or flow conditions can significantly change patterns of deposition and erosion. Cohesive and noncohesive sediment shares this transport dependence upon the balance of bed shear against sediment supply, but differs in that noncohesive sediment beds may experience simultaneous erosion and deposition of particles while cohesive sediment beds generally do not. They also differ in that available

noncohesive sediment tends to be depositable, while available cohesive sediment may not be, requiring a certain level of aggregation in order to become depositable.

53. In estuarine areas where fine-grained sediment is concentrated, the amount of sediment in suspension may become quite large, on the order of thousands of parts per million. Stratified suspensions may occur with multiple sharp vertical gradients in concentration. As long as the high concentration sediment moves with the water or under its own weight as a density current, it is termed mobile suspension. When interaction between particles causes the suspension to stop moving, it becomes a stationary suspension, termed fluid mud, fluff, or sling mud. At this point, the sediment suspension has a density only slightly greater than that of water and may gradually consolidate to become part of the bed if not disturbed; however, it possesses a relatively low strength and may be eroded en masse.

Redeposition

54. If mound dynamics calculations determine that sediment will be eroded from an DMADS, the destination of that sediment must be determined. A numerical or physical modeling approach to the problem will provide an indication of the amount eroded and the ultimate fate of the resuspended sediment. The use of field observations to determine the fate of resuspended sediment is rather difficult. The depths of deposits are seldom large enough for accurate measurement by standard hydrographic surveying methods. Sedimentation stakes, deposition pits, and a photographic technique known as REMOTS (Remote Ecological Monitoring of the Seafloor) can be used, but they require either a foreknowledge of where sediment will travel or a rather extensive (and expensive) sampling network.

55. An alternative to hydrographic surveying is sediment-tracer tests. Either naturally occurring tracers, such as mineralogically identifiable grains, or artificial tracers, such as radionuclides, can be tracked to detect areas and quantities of deposition.

56. Analytical techniques for predicting the long-term fate of resuspended sediments are extensions of the work used to determine how much sediment leaves the site. Knowledge of hydrodynamic conditions at the site is required to calculate directions and rates of transport out of the site. Hydrodynamic conditions along the projected path may then be predicted so that zones of deposition can be identified. In tidal flows, sediment may deposit

temporarily and be resuspended again during strength of flow. Because of the oscillatory nature of the flow, the resuspended sediment may move in and out of the disposal site or may simply oscillate. Describing this process analytically requires a more detailed knowledge of the disposal site's hydrodynamic conditions than is usually available for an analytic study. Such an effort is often reduced to computing a path, assuming a distribution, and concluding that the redeposited sediment thickness is either negligible or significant.

57. Material redeposition occurs when the supply of depositable sediment exceeds the flow's capability to transport it; thus, an area in sedimentation equilibrium is one in which the flow is just able to prevent quasi-permanent deposition of the available supply of sediment. A change in material redeposition behavior at a site can be the result of an increase in depositable sediment and/or a decrease in transport capability due to changed environmental forcing conditions.

58. Flow transport capacity is a function of the strength of the current and its turbulent fluctuations. Noncohesive sediment transport functions use the flow velocity raised to a power to compute transport rates; many use bed shear stress (proportional to velocity squared) to determine conditions of incipient motion. For cohesive sediment, bed shear stress is used to compute the rate of deposition and erosion. Thus, for both cohesive and noncohesive sediment flow, transport capability can be related to a characteristic current velocity.

59. The depositable sediment supply to an area is determined by the shear stress values in the area, the amount of sediment that is delivered to the area, the amount of sediment available on the bottom nearby, and (in the case of cohesive sediment) the intensity of aggregation processes. For example, the depositable sediment supply will be increased by introduction of sediment-laden waters, erosion of channel sides or bottoms, trapping supplied sediment (i.e., turbidity maxima), or an increase in aggregation of cohesive sediment that is already present. Aggregation is increased by the increased rate of particle collisions due to more internal flow shear, by obstacle-induced turbulence, or by water quality changes that facilitate physico-chemical bonding.

PART III: METHODS OF ANALYSIS FOR MOUND RESUSPENSION AND DYNAMICS

60. The first section of this part discusses methods for estimating conditions that are sufficient to cause sediment erosion or resuspension. The remaining four sections discuss the methods for determining mound resuspension and dynamics: steady-state analytical methods; time- and rate-dependent analytical methods; modeling; and field and laboratory studies. Appendixes A-D provide additional information on field and laboratory studies and modeling.

Requirements for Resuspension

61. The resuspension and transport of movable material (cohesive or noncohesive) is functionally related to the availability of physical stresses (forces) that induce such movement. If the natural environmental forces existing in the region are less than those forces required for particle motion, the material will remain at rest. If, however, the available stresses are greater than those required to place material in motion, then motion will be initiated. For noncohesive sediment, the smaller particles will begin to move prior to motion by larger particles. When this happens, the surface area of the disposal mound becomes armored with slightly larger sized material, and the forces (stresses) necessary to initiate motion of this slightly larger material are thereby increased accordingly. For cohesive sediment, shear strength increases with depth in the bed so stresses necessary to initiate motion increase with erosion. Thus, for a given intensity of available stresses, the rate of material movement decreases temporally. This implies that the surface area of the mound (the spatial extent) is fundamentally related to the rate at which material can be resuspended and transported from the disposal site. However, a smaller area one-half the size of a larger area will not necessarily lose 50 percent by volume of the material lost by the larger dump site. Other inherent factors regarding boundary layer motion induce sliding and rolling movements of particles on intermittent bases that are related to the turbulence associated with the currents and wave motion. If knowledge regarding the magnitudes of the physical forces of the region does not exist, a suitable, comprehensive field data-collection program should

be initiated to obtain those data necessary for performing the appropriate analyses. All such data are site specific.

Stresses that initiate resuspension

62. The drag forces exerted by a slowly moving liquid on a boundary are determined by the viscosity of the liquid. At higher rates of flow, there is added transfer of momentum normal to the boundary that is greater than molecular transfer; the flow changes to turbulent; and the added momentum transfer can be expressed as an eddy viscosity that is much greater than the fluid viscosity. Most flows that occur in open waters are turbulent, where this term denotes motion in which irregular fluctuations are superposed on the main flow. These fluctuations are so complex that they are not readily amenable to mathematical treatment; however, their effects are as if the viscosity of the liquid were increased by factors of hundreds or even thousands (Schlichting 1968). Time averages of the turbulent motion must be considered in an analysis of sediment movements. Mean values are taken over a sufficiently long interval of time for them to be largely independent of time.

63. The transport of sediment or dredged material under the simultaneous action of waves and currents is related to the magnitude of the shear stresses exerted by the combined wave and unidirectional current motions. The unidirectional currents that are superposed on the wave-induced currents may be tidal currents or other currents of local nature.

64. Tidal and other unidirectional currents. The drag and lift forces for tidal or other unidirectional currents, when expressed in the usual manner, are:

$$F_D = \frac{1}{2} C_D \rho A u_b^2 \quad (1)$$

and

$$F_L = \frac{1}{2} C_L \rho A u_b^2 \quad (2)$$

Here, fully turbulent flow is assumed and the stresses (forces) are related to the square of the velocity, u_b , near the bed. (The density of water is expressed by ρ .) Unfortunately, the values of the drag and lift

coefficients, C_D and C_L , respectively, are not well known. Furthermore, the projected area, A , of the particles depends upon the shape of the elements, and this incorporates additional uncertainties into the analysis. Experimental verification to obtain these empirical quantities has not been entirely successful, due in part to the inability to obtain good definition of the bottom velocity, u_b . Hence, the theoretical applications of the bottom velocity analyses are of limited use from a practical standpoint, and most researchers tend to use the average velocity of flow, U_C . The total equivalent force due to the current becomes:

$$F_C = \frac{1}{2} C_C \rho U_C^2 \quad (3)$$

where the uncertainties associated with the particle projected area are incorporated inherently into the effective current coefficient, C_C .

65. For a control-volume approach to the flow analysis, the gravitational component in the direction of flow is balanced by the frictional resistance at the bed, and the resulting equality yields the bottom shear stress:

$$\tau_o = \gamma DS \quad (4)$$

where

γ = weight of the liquid per unit volume

D = depth of uniform flow

S = slope of the hydraulic gradient producing the flow

When incipient motion of the bed particles occurs, the resulting shear stress is considered to be the critical incipient shear stress, τ_i . Defining the friction velocity as

$$u_* = \left(\frac{\tau}{\rho} \right)^{1/2} \quad (5a)$$

it is possible to establish a reference dimensionless quantity incorporating both fluid properties and sediment size, referred to as the shear Reynolds number, Re_* :

$$Re_* = \frac{u_* d}{\nu} \quad (5b)$$

where

d = particle diameter

ν = kinematic viscosity of water

66. Graf (1971) reports that Shields (1936) appears to have been the first in the field of sediment-transport mechanics to have used this concept for the special case of uniform sand. Since that time, however, other researchers have experimented with a wide assortment of materials with densities ranging from 1.06 to 7.90 g/cu cm. Their results are shown in Figure 6, which is a representation of the critical bottom shear stress, τ_1 , as a function of the shear Reynolds number. Considering all the experimental data (which were obtained from different researchers and with quite different experimental arrangements) that went into developing this figure, the overall scatter should not be alarming. Other threshold curves that form the basis for predicting initial sediment motion, knowing some combination of sediment grain size and density or water velocity and stress, have been extrapolated from the Shields diagram; however, this work for the initiation of sediment movement under tidal or other unidirectional currents serves as a general criterion.

67. It is apparent that loose fine sand erodes the easiest and that the increased resistance to erosion for extremely small silt-clay particles must be attributed to the presence of cohesive and adhesive forces. Postma (1967) expanded the work of various authors (Figure 7) and showed that cohesive fine sediment gradually loses water, consolidates with time, and becomes more resistant to erosion. When suspended particles are deposited by a decelerating current, they will be graded from sand to silt to clay. However, when sediment consisting of less than 60 percent water is eroded, the fine sand will be resuspended at lower current speeds than clay or coarse sand.

68. Wave-induced stresses. In steady, turbulent flow, it is known that shear stresses at the bottom are directly proportional to the square of the average velocity. This analogy has been extended to oscillatory flows by Jonsson (1965, 1978) to relate the maximum bottom shear stress and the maximum free stream velocity through a coefficient, f_w , termed the wave friction factor, such that

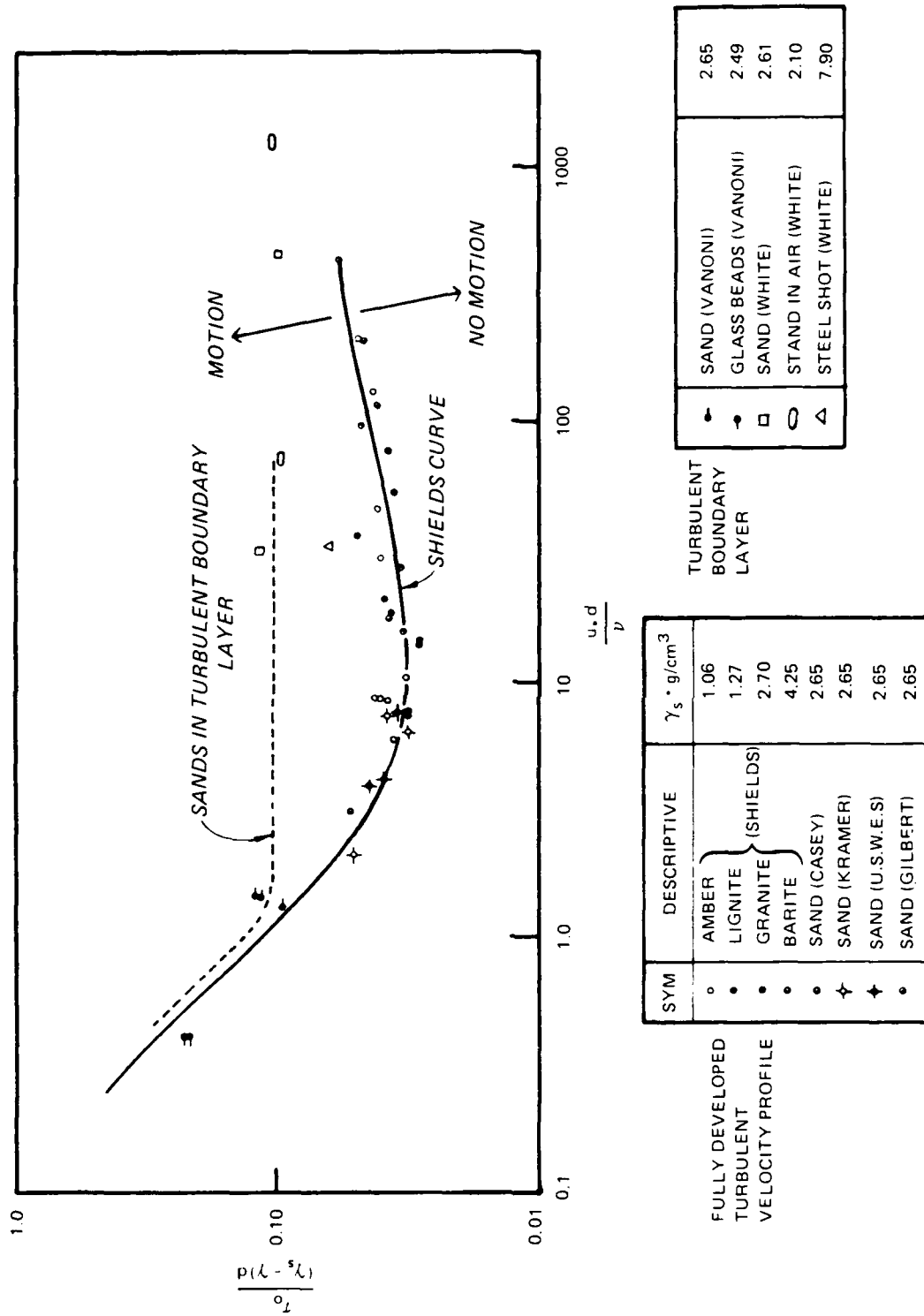


Figure 6. Shields diagram of dimensionless critical shear stress versus shear Reynolds number (after Basco et al. 1974)

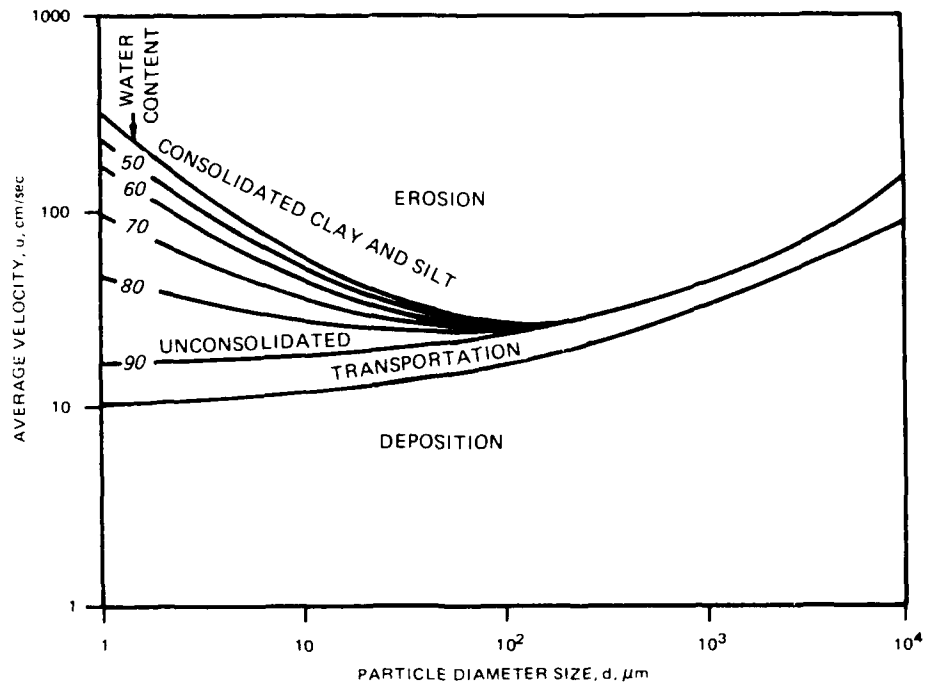


Figure 7. Erosion-transport-deposition criteria for different grain sizes (after Basco et al. 1974)

$$\tau_{\max} = \frac{1}{2} f_w \rho u_{\max}^2 \quad (6)$$

69. For laminar boundary layers, the wave friction factor can be deduced as

$$f_w = \left(\frac{2}{RE} \right)^{1/2} \quad (7)$$

where RE is the wave Reynolds number, defined as

$$RE = \frac{u_{\max} a_o}{\nu} \quad (8)$$

Here u_{\max} is the maximum particle velocity at the bottom and a_o is the orbital diameter that now becomes the characteristic length scale of the viscous motion. According to Jonsson (1965), the flow remains laminar in the boundary layer only as long as RE is less than about 12,600. By this criterion, practical cases of wave motion at disposal sites will be turbulent.

70. The wave friction factor concept has been used to present empirical data obtained by various investigators, by extending the analogy of steady-state friction factors for pipe flow as reflected in the Moody diagram. Jonsson (1965) found the wave friction factor f_w to be a function only of the relative roughness a_o/d_e for fully rough turbulent flow in the boundary layer, where d_e is the effective grain size on the bed or bed roughness. His wave friction factor diagram can be used to determine the maximum shear stress exerted by specific wave conditions if the value of the relative roughness is known. This maximum bottom shear stress associated with the wave motion is generally considerably larger than the shear stress from a steady current of the same magnitude. This emphasizes the importance of wave motion as an initiating agent that makes sediment available for transport by a current that would otherwise not be able to initiate sediment motion.

71. The wave friction factor diagram was first represented by Jonsson (1965), but it was based on very few data. Riedel et al. (1972) performed systematic experiments on both smooth and sand-roughened beds in an oscillating water tunnel to measure the maximum shear stress from which the wave friction factor was then computed. These data were sufficient to define the wave friction factor over the range of practical use, which is presented in Figure 8 in the same form as that of Jonsson (1965). The equivalent sand roughness for a range of grain size particles was determined to be:

Sand Size d_{50} , mm	Effective Sand Roughness, mm
0.37	1.41
1.65	8.43
3.13	15.80
9.80	51.50
50.00	139.00

72. Riedel et al. (1972) concluded that, within the laminar range, the agreement between theory and experiment was very good. The upper limit of the laminar range occurs for $RE \approx 10^4$, which corresponds approximately to the middle of the observed range of values of transition for wave flume tests. In the smooth turbulent range, the data points define a curve that lies approximately 25 percent below the predicted line. However, this difference is

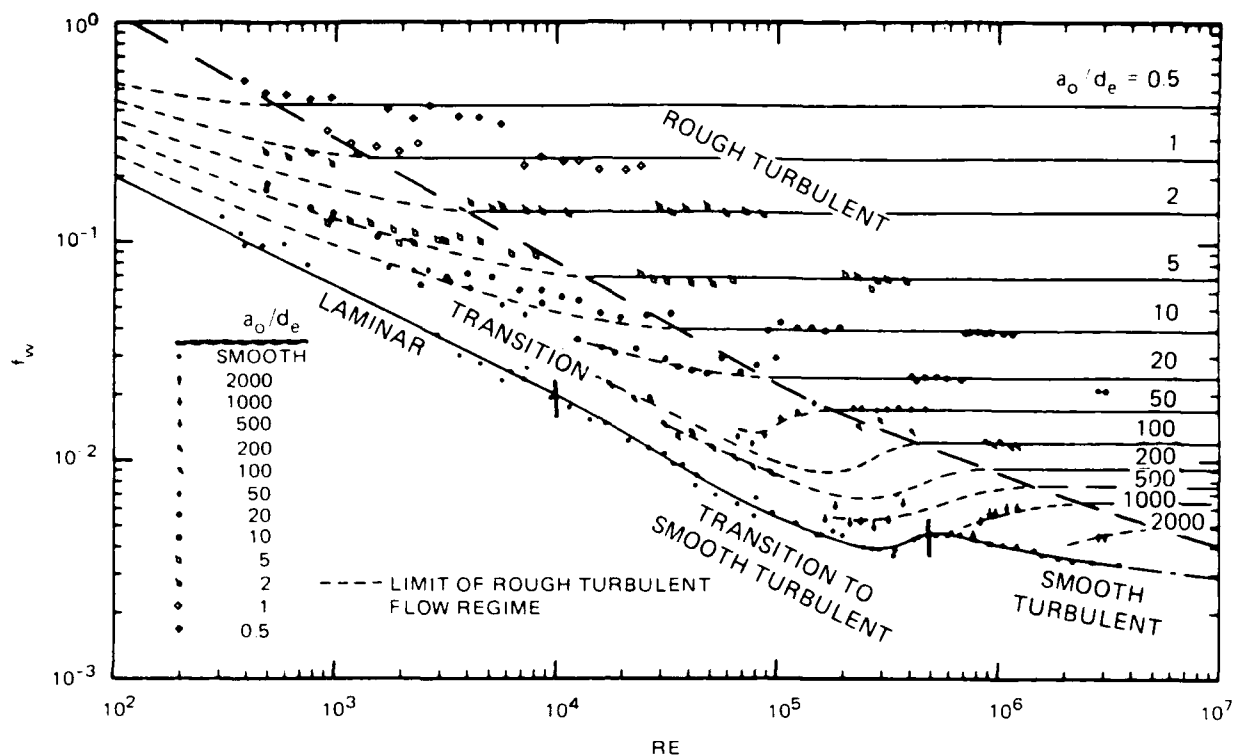


Figure 8. Experimental wave friction factor, f_w (after Riedel et al. 1972)

remarkably small considering the assumptions that were made in the derivation of the theoretical expression. The lower limit of the smooth turbulent range was found to be $RE = 6 \times 10^5$, which agrees quite well with the theory.

73. For the rough turbulent flow regime, that most commonly found in the open water, RE is no longer important, and a least-squares fitting technique relates wave friction factor, f_w , to roughness as follows:

$$\left(\frac{1}{4.95 f_w^{1/2}} \right) + \log \left(\frac{1}{4 f_w^{1/2}} \right) = 0.122 + \log \left(\frac{a_o}{d_e} \right) \quad (9)$$

This expression is consistent with the assumption that a logarithmic velocity law exists near the bed. For $a_o/d_e > 25$, the orbital amplitudes are relatively large, and unidirectional flow is approached; the phase difference between the free stream velocity and the shear stress at the bed approaches zero. Hence, it was concluded that the assumption of a logarithmic velocity profile at the instant of maximum free stream velocity was reasonable.

74. The maximum value of the instantaneous bed shear stress under oscillatory wave motion has been utilized by Nielsen (1979) to evaluate the magnitude of the wave boundary layer friction factor f_w in terms of the periodic characteristics of the wave motion. The maximum value of the shear stress can be expressed as

$$\tau_{\max} = \frac{1}{2} \rho f_w (A\omega)^2 \quad (10)$$

where velocity of the water particle just outside the boundary layer is given as

$$u(t) = A\omega \sin \omega t \quad (11)$$

with the wave amplitude defined as A and the wave frequency expressed as ω . Since τ_{\max} is proportional to the square of the water velocity, the variation of τ with time can be considered as

$$\tau(t) = \tau_{\max} | \sin \omega t | \sin \omega t \quad (12)$$

75. For laminar flow, the friction factor for wave motion is:

$$f_w = 2 \left(\frac{\nu}{A^2 \omega} \right)^{1/2} \quad (13)$$

but for the more general turbulent cases, it was found convenient by Nielsen (1979) to use Swart's (1974) formula:

$$f_w = \exp \left[5.213 \left(\frac{d_e}{A} \right)^{0.194} - 5.977 \right] \quad (14)$$

which is a good approximation to Jonsson's (1965) semiempirical expression. Equation 14 is applicable when the ratio of wave amplitude to roughness of the bed exceeds about 1.7. For smaller ratios, f_w has been found to remain constant at 0.28. Under these considerations, Equation 9 can be rearranged as

$$\frac{1}{4(f_w)^{1/2}} + \log \frac{1}{4(f_w)^{1/2}} = -0.08 + \log \frac{A}{d_e} \quad (15)$$

and is displayed in Figure 9.

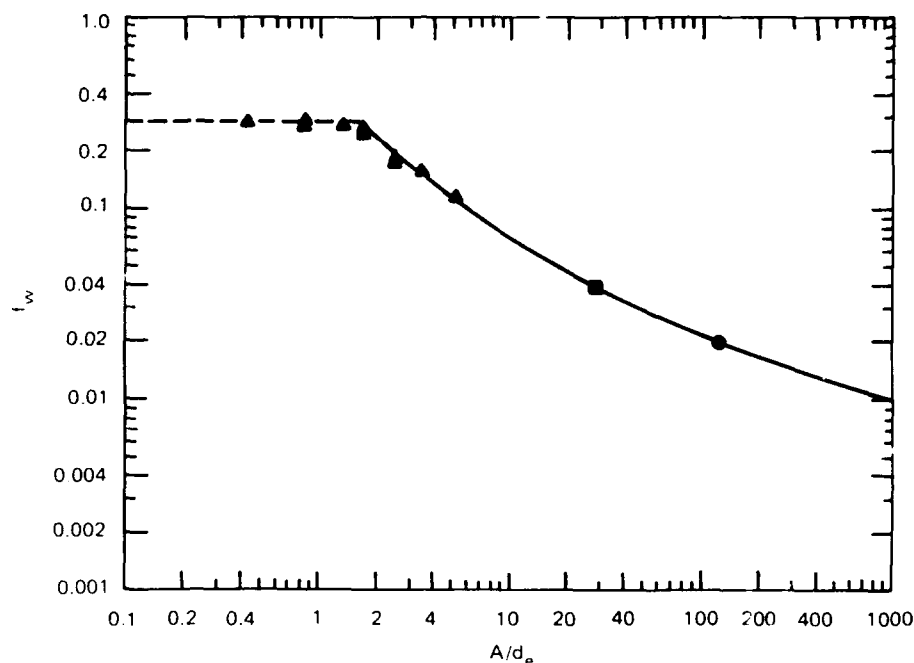


Figure 9. Wave friction as a function of wave amplitude to bed roughness ratio (after Nielsen 1979)

76. The bed roughness was also investigated by Nielsen (1979) and found to be equal to the grain diameter when the bed is flat and well smoothed. When the bed is not so well smoothed, it was found reasonable to use

$$d_e = 2.5d \quad (16)$$

where d is grain diameter. When the bed is covered by ripples, the total shear stress was found to be a function of both the ripple height, Δ_r , and the ripple length, λ_r , as

$$d_e = 25 \frac{(\Delta_r)^2}{\lambda_r} \quad (17)$$

77. Considerable attention has been given to the threshold of sediment motion under unidirectional currents. Far less attention has been given to the threshold of sediment motion under oscillatory wind waves because of the greater measurement difficulty under the wave motion where currents are continuously varying and accelerations are important. Historically, the procedure has been to apply the Shields diagram (Figure 10) for unidirectional flow to the case of wave motion and to realize there might be some error since accelerating currents exert a greater shear stress than does a steady flow of the same magnitude.

78. Komar and Miller (1974) and Madsen and Grant (1975) independently investigated five sets of previously published data to determine the threshold of sediment motion under wave action. These included data by Bagnold (1946), Manohar (1955), Eagleson et al. (1958), Horikawa and Watanabe (1968), and Rance and Warren (1968). It was found that the data are in good agreement with the Shields function when plotted on a Shields diagram of bed particle Reynolds number as a function of the parameter:

$$\frac{1}{\psi} = \frac{\tau_{\max}}{(\rho_s - \rho) g d} \quad (18)$$

where τ_{\max} is the maximum bottom shear stress. Figure 9 is the display of all these data in the form of the customary Shields diagram that includes the influence of the wave friction factor, f_w (see Jonsson 1965, 1978). Therefore, Madsen and Grant (1975) concluded that with all its limitations, the Shields function may serve as a relatively reliable and quite general criterion for the threshold of sediment movement under water waves.

79. Komar and Miller (1974) recognized that, for oscillatory wave motion, the threshold of movement for a given grain diameter d can be specified by a wave period T and orbital velocity at the near bottom, u_{\max} . The best-fit expression for this relation is:

$$\text{for } d < 0.05 \text{ cm: } \rho \frac{u_{\max}^2}{(\rho_s - \rho) g d} = 0.21 \left(\frac{a_o}{d} \right)^{1/2} \quad (19)$$

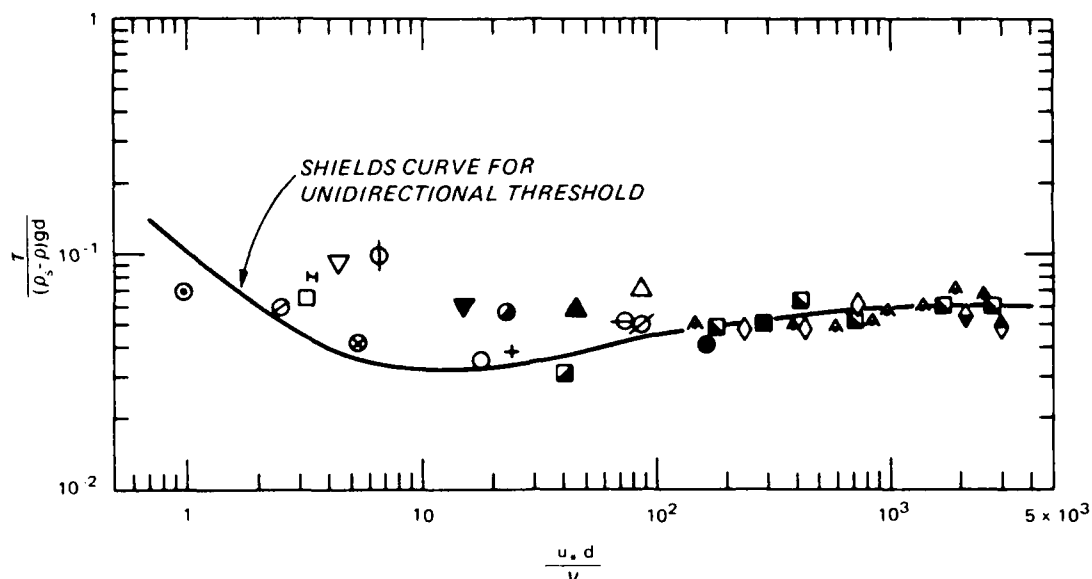


Figure 10. Comparison between the Shields curve for threshold under unidirectional currents and threshold for oscillatory wave motion (after Komar and Miller 1974)

$$\text{for } d < 0.05 \text{ cm: } \rho \frac{u_{\max}^2}{(\rho_s - \rho)} gd = 0.46 \pi \left(\frac{a_o}{d} \right)^{1/4} \quad (20)$$

Equations 19 and 20 are displayed graphically in Figure 11. Since all the data used in these developments came from laboratory experiments of various configurations, probably the most severe shortcoming of these results is the limited amount of prototype field data to verify these conclusions regarding the threshold of sediment motion under oscillatory wave motion.

80. Interactions of wave trains of differing periods under natural open-ocean conditions may generate instantaneously higher velocities, and sediment motion may occur at lower velocities than those implied in these analyses. Dingler and Inman (1976) studied ripples in fine sand in the field at La Jolla, Calif. They found that when the bottom is subjected to waves of variable height and of sufficient intensity that transition ripples occur following the passage of intermediate height waves, the larger waves will produce sheet flow and a flat bottom. As the wave conditions intensify, sediment begins to move and vortex ripples start to form. The ripples eventually reach an equilibrium configuration with a steepness of about 0.15.

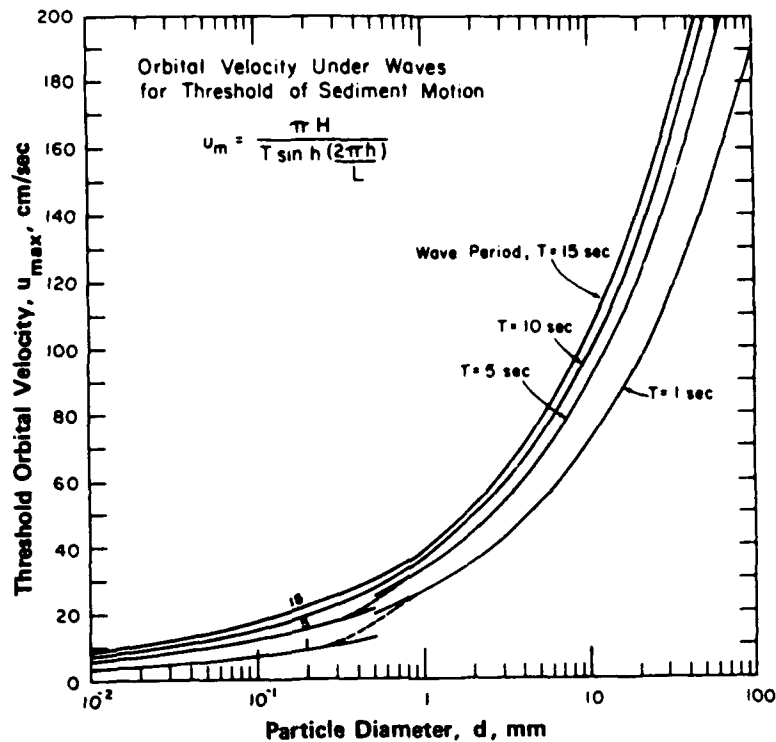


Figure 11. Near-bottom orbital velocity for threshold of sediment motion under oscillatory waves
(after Komar and Miller 1974)

81. Data from previous wave motion studies have been utilized by Lenhoff (1982) to establish a generally applicable criterion for the onset of grain motion under oscillatory flow. The problem is a complex one, mainly due to the numerous variables involved. Therefore, most authors prefer to base their criteria on experimental rather than purely theoretical analyses.

82. Since the ocean bottom is normally rippled in the areas of coastal engineering interest, any incipient motion criterion should extend from flat bed to ripple bed conditions. The parameters that were chosen for the graphical presentation of the data reanalyzed by Lenhoff (1982) are the shear Reynolds number, R_* , and a dimensionless grain diameter, D_* , where

$$R_* = \frac{u_* D_{50}}{\nu} \quad (21)$$

and

$$D_* = \left(\frac{\Delta_s g}{\nu^2} \right)^{1/3} D_{50} \quad (22)$$

with

$$u_* = \left(\frac{\tau}{\rho_w} \right)^{1/2} \quad (23)$$

$$\tau = \frac{1}{4} \rho f_w U_o^2 \quad (24)$$

where

$$\Delta_s = (\rho_s - \rho) / \rho$$

u_* = shear velocity

ν = kinematic viscosity

τ = mean shear stress

f_w = wave friction factor

U_o = maximum orbital velocity for waves at the bed

83. The wave friction factor, f_w , can be found from empirical relationships (Riedel et al. 1972) as a function of orbital Reynolds number and the ratio a_o/r :

$$f_w = f \left(RE, \frac{a_o}{r} \right) \quad (25)$$

where

a_o = maximum orbital excursion from mean position by the motion of particles at the bed (orbital diameter)

r = bed roughness

84. The generalized procedure of analysis may therefore be summarized as follows:

- a. Computation of wave properties U_o and a_o .
- b. Computation of the bed roughness, r , using the following formulas:

$r = 2D_{90}$ for a flat bed (Kamphuis 1975)

$r = 25(\Delta_r)^2/\lambda_r$ for rippled bed (Swart 1976)

where

D_{90} = grain size from which 90 percent of material is finer

Δ_r = ripple height

λ_r = ripple length

c. Computation of a_o/r .

d. Computation of orbital Reynolds number: $RE = U_o a_o / \nu$.

e. Determination of the wave friction factor, f_w , using empirical relationships based on more than 600 data sets (Riedel et al. 1972).

f. Calculation of R_* and D_* .

85. A total number of 643 data points consisting of combinations of R_* and D_* at the onset of movement were calculated in the above manner by Lenhoff (1982). Figure 12 is a logarithmic plot of the results, together with a best-fit curve through the data points. This empirical curve was found to be very closely simulated by the parabolic equation:

$$\log_{10} R_* = 0.092 (\log_{10} D_*)^2 + 1.158 \log_{10} D_* - 0.367 \quad (26)$$

86. The conditions for movement under unidirectional steady flow have been widely studied by numerous researchers, and well-established criteria for the beginning of movement exist. The works by Shields (1936) and Ackers and White (1973) are probably the most widely used and acknowledged in this regard. A direct comparison between these two criteria and the results of the study by Lenhoff (1982) can be made. Figure 13 shows the original Shields (1936) and Ackers and White (1973) data points, together with the curve defined by Equation 26. The envelope encompasses data scatter for oscillatory flow. The unidirectional data can be seen to fall well within the scatter of the oscillatory-flow data points. It can, therefore, be concluded that there is no significant difference between the criterion for uniform flow and the criterion of oscillatory flow as derived in this study. A single criterion can be applied with the same degree of accuracy for both flow conditions. Consequently, it is reasonable to assume that the criterion developed by Lenhoff (1982) can be applied for wave-generated flows, current-dominated

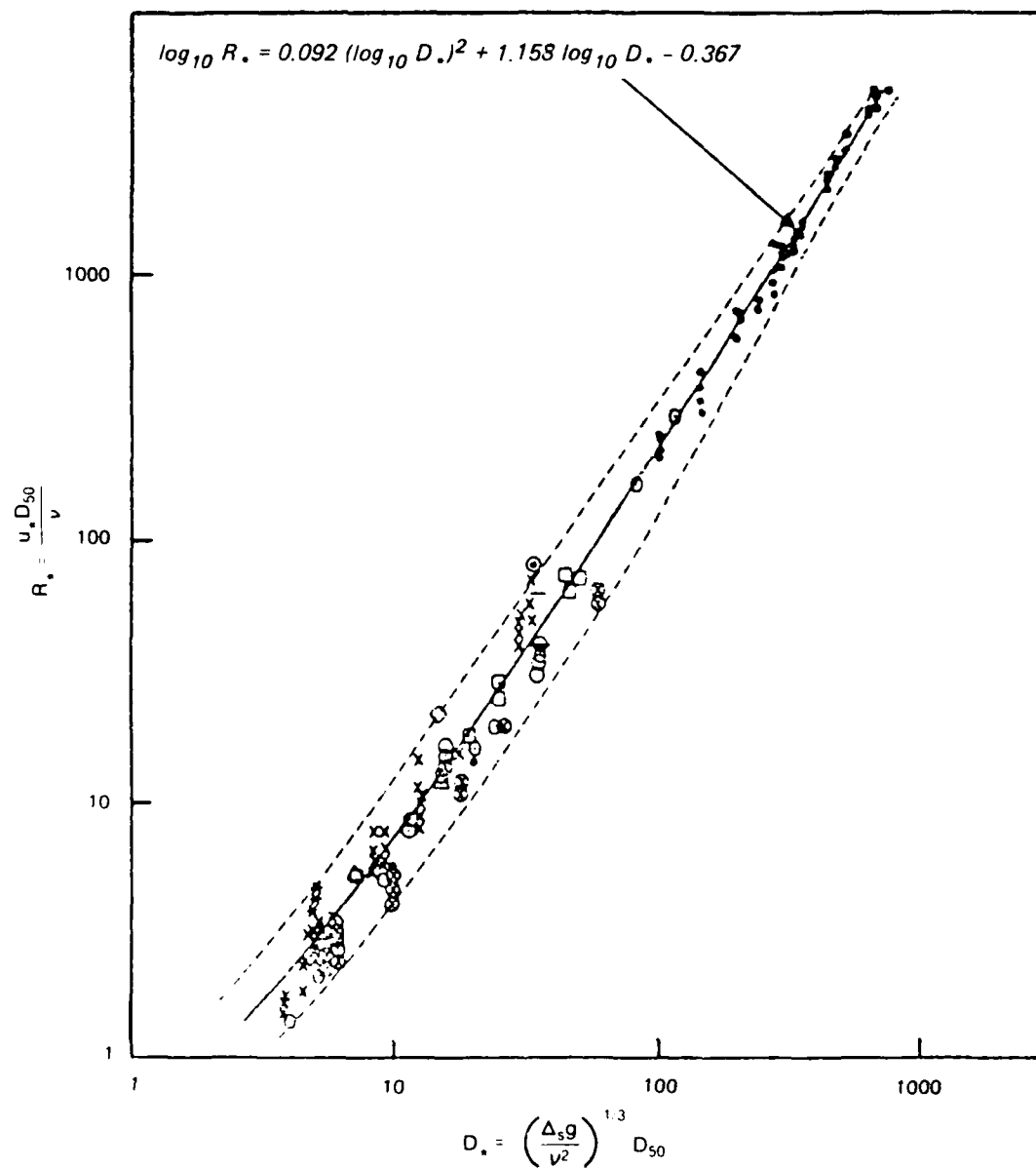


Figure 12. Incipient motion of sediment particles under oscillatory flow, experimental results from various sources (after Lenhoff 1982)

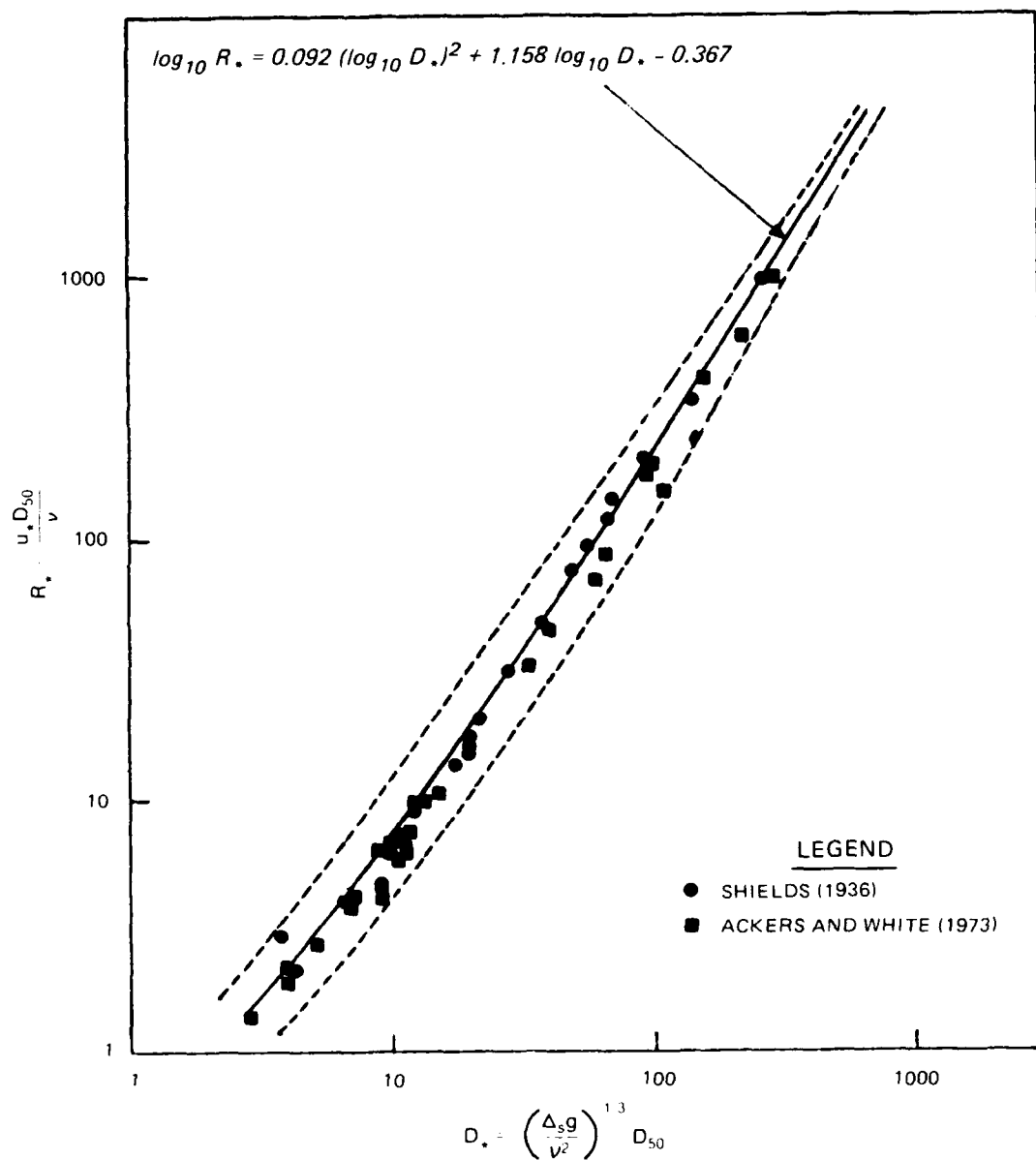


Figure 13. Comparison of incipient motion of sediment particles under oscillatory-flow criteria with unidirectional flow criteria (after Lenhoff 1982)

flows, and all combined flow modes in between, provided the correct definition of the shear velocity, U_* , is used in Equation 21.

87. Conclusions drawn from this investigation by Lenhoff (1982) include:

- a. An empirical criterion, Equation 26, was derived for the onset of grain motion on either a flat or a rippled bed under the influence of wave action.
- b. It was shown that the new criterion is not only valid for incipient motion due to waves, but also does not differ from widely used incipient-motion criteria for steady-state conditions. In addition, provided the correct definition of the bed shear stress is used, it also applies to the case of combined current and wave action. It can, therefore, be seen as a fairly universal criterion for the onset of grain movement.
- c. Comparison to sediment load data via the adapted Ackers and White sediment-load formula given by Swart and Fleming (1980) yielded values of the incipient motion mobility number that would have been required to allow the sediment load formula to predict the measured sediment load. These indirectly calculated incipient motion data fall within the scatter of the original data used to derive Equation 26. This serves as a very strong independent proof of the adapted Ackers and White sediment load formula given by Swart and Fleming (1980).

88. Combined stresses due to unidirectional currents and waves. Thomas and McAnally (1985) defined the shear velocity, U_* , as

$$U_* = \sqrt{\frac{1}{2} f_c U^2 + \frac{1}{4} f_w U_o^2} \quad (27)$$

where

f_c = shear stress coefficient for currents, 0.0003 (Sternberg 1972)

U = current velocity

f_w = shear stress coefficient for waves

U_o = maximum orbital velocity for waves at bottom

89. The actual bed shear stress, τ_b , is related to the shear velocity, U_* , as

$$\tau_b = \rho U_*^2 \quad (28)$$

Sleath (1978) had displayed the data previously analyzed by Komar and Miller

(1974) by computing a nondimensional grain size, D_* , for a particular grain-size diameter, D :

$$D_* = \left[\frac{(\rho_s - \rho)g}{\rho v^2} \right]^{1/3} D \quad (29)$$

90. D_* is functionally related to a critical dimensionless shear stress parameter, ψ_c , defined as

$$\psi_c = \frac{\tau_c}{(\rho_s - \rho)gD} \quad (30)$$

This relationship is displayed in Figure 14. For a specific grain-size fraction, D , in the bed, the dimensionless grain size, D_* , is determined from Equation 29 and the dimensionless critical bed shear stress, ψ_c , is determined from Figure 14. By knowing the dimensionless critical bed shear stress, the critical bed shear stress for material motion, τ_c , is calculated from Equation 30. Hence, one can readily determine whether the stresses are great enough to cause material to move by comparing τ_b with τ_c .

Determination of wind-wave characteristics

91. The water-wave contribution to sediment transport along the bottom is directly related to characteristics of the wave climate, such as wave height, wave period, and wave direction of propagation. Prior to the early 1980s, these characteristics of surface gravity waves on water were not generally well known. Some specific locations along the coastline of the United States had been gauged with wave-measuring instrumentation for limited periods of time, but such direct measurement of water-surface time-histories was minimal. Wave hindcasting from synoptic weather charts had been conducted for some coastal regions, but these hindcasting efforts were either of very short record duration (usually 3 years of weather records), or the methodology suffered from extensive numerical generation shortcomings, including the application of a singular wave model for the determination of wave statistics. Most knowledgeable researchers agree that the spectral approach is significantly better, and, indeed, in late 1976 WES initiated a study to provide,

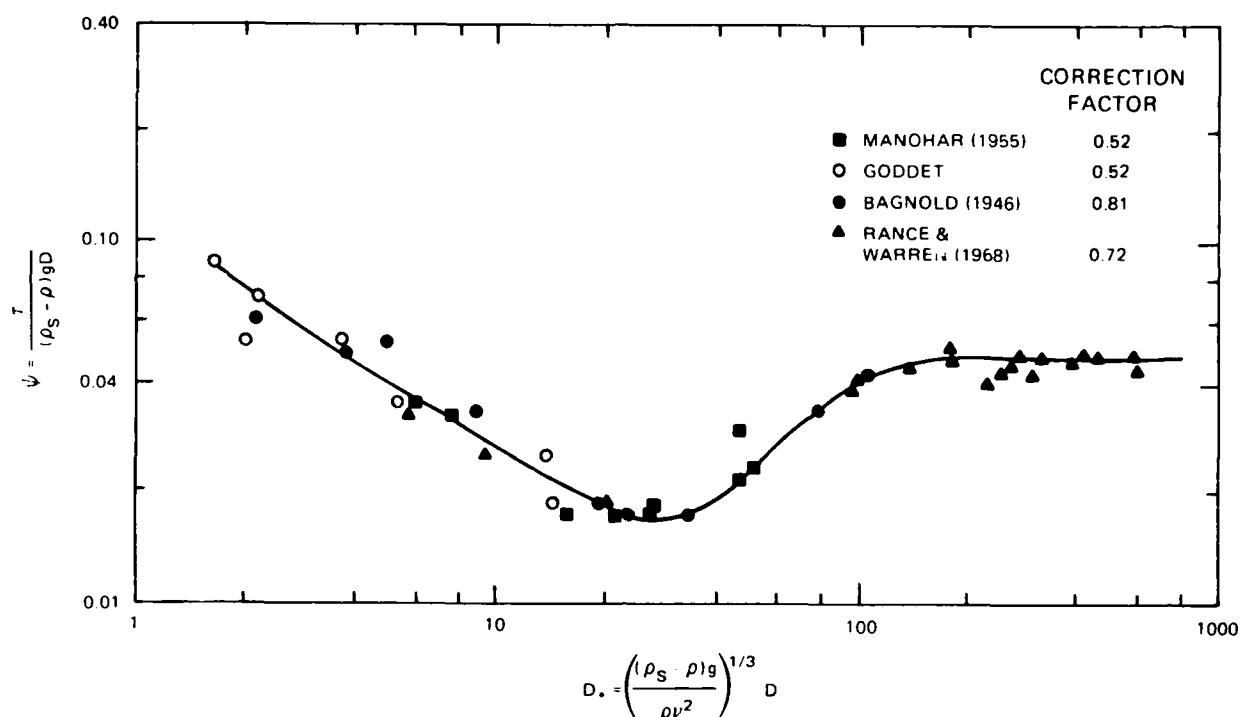


Figure 14. Modified Shields curve for threshold of sediment motion under oscillatory wave motion from various sources (after Sleath 1978)

through hindcasting, a directional spectral wave climatology for all continental US coastlines (Atlantic, Pacific, Gulf of Mexico, and Great Lakes). This climatological information is to be produced by a Wave Information Study (WIS) that will generate numerical simulation of wave growth, propagation, and decay under historical wind fields of 20-years duration (1956-1975).

92. The final product of this wave hindcasting system is a voluminous data base of wave-parameter data organized by site and time interval. To provide access to this database for Corps field offices, a computer-based system for storage, retrieval, and computation is operational, the Sea-State Engineering Analysis System (SEAS) (Ragsdale 1983).

93. The SEAS is capable of performing a number of statistical analyses in the form of various report texts using source input from an extensive database of numerically simulated hindcast wave data. Data are retained on off-line media (magnetic tape) for economy and are selectively loaded to user-designated files from which the data can be accessed more efficiently. Wave data are segregated by oceanographic region (e.g., Atlantic Ocean) with

each region consisting of a number of sites (also known as stations) for which readings have been obtained. Data for each station are ordered chronologically. The SEAS data base now contains data for 252 Atlantic Ocean stations and 80 Pacific Ocean stations, encompassing a 20-year period from January 1956 through December 1975 (at 3-hr intervals).

94. WIS separated the wave climatology into three phases:

- a. Phase I: Numerical hindcast of deepwater wave data from historical surface pressure and wind data.
- b. Phase II: Numerical hindcast at a finer scale than Phase I to better resolve the sheltering effects of the continental geometry. Phase I data serve as the boundary conditions at the seaward edge of the Phase II grid.
- c. Phase III: Transformation of Phase II data into shallow water and inclusion of long waves.

Phases I, II, and III for the Atlantic coast have been completed and are available on SEAS. Phase III, which contains frequency of occurrence of various waves for height, period, and direction for 166 nearshore locations along the Atlantic coast of the United States, is also available as WIS Report 9 (Jensen 1983). Phases I and II have been completed for the Pacific coast and are on SEAS. Phase III for the Pacific coast is completed north of Point Conception. Numerical models for transferring the deepwater wave climates past the sheltering offshore islands of southern California are being developed. Only Phase II wave information is being produced for the Gulf of Mexico. Wave data for the Great Lakes will be available by the end of 1987.

Steady-State Analytical Methods

95. Because of the spatial and temporal aspects of dredged material disposal and the settling, resuspension, and potential transport from the site location, many of the theoretical studies regarding these phenomena have been adapted for solution by numerical simulation models on high-speed large-capacity computer systems. Analytical methods are considered to be those mathematical expressions (equations) that can be solved in closed form to provide solutions (estimates). Such analytical methods which have been developed with specific reference to DMADS are relatively limited.

96. Steady-state refers to those physical processes that do not vary with time (e.g., flow in a pipe under constant pressure) or to physical

processes that, when averaged over a sufficiently long period, are independent of time (e.g., wind waves at a specific location averaged over a year). In the first instance (flow in a pipe), the processes are truly constant, steady, and nonvarying with time. In the second instance (wind waves on the ocean), they vary temporally but, if averaged over a period of 12 months, would provide a yearly average value that would approximate a constant value regardless of the year in which the average value was determined.

97. Steady-state analytical methods for the analysis of mound resuspension, therefore, are solutions to those mathematical expressions that can be solved in closed form to provide solutions to processes at DMADS when the ambient parameters of currents, waves, disposed material, and disposal operations have been resolved to essentially constant conditions. Steady-state analyses require: the selection of so-called "representative" energy levels of currents and waves; combining these representative energy levels; representation of sediment characteristics; knowledge about the disposal mound configuration; assumptions regarding disposal processes; understanding the threshold of sediment movement; estimates of the erosion rates of disposal mounds; and knowledge of the effects of the disposal mounding on ambient current conditions.

Representative
average steady-state values

98. Energy levels of currents. Evaluation of currents for a steady-state analysis should take into account the variation of tidal currents over a complete tidal cycle and over a range of tidal amplitudes in order to properly characterize them. When possible, the behavior of near-bed current velocities should be emphasized. The representative velocity of tidal, riverflow, and longshore currents will usually not be a simple arithmetic mean, but rather a value that characterizes the average from the minimum at which incipient motion occurs to the maximum value. In some cases, an average of velocity to some power is appropriate.

99. Energy levels of waves. Knowledge about wave characteristics on the open ocean can be ascertained for monthly frequencies of occurrence by direction from various wave hindcast studies or prototype wave-gaging programs. The average percentage of time that waves exist with periods and heights sufficient for causing substantial motion on the bottom can be

ascertained. From such studies, the representative wave condition that can simulate an average steady-state wave climate can be determined.

100. Combining energy levels of waves and currents. Linear addition is appropriate when more than one current exists, and the currents are essentially unidirectional (e.g., tidal current and riverflow in an estuary). Vectorial addition has been used to combine wave and current velocities into one representative value to be applied for shear-stress computations. Agitation-transport evaluations have been performed where the magnitude of the wave-induced current is used only to determine if the bottom material can be moved by this wave stress, and then material transport is deduced by a consideration of the tidal or other unidirectional current magnitude.

101. Sediment characteristics. The characterization of each solid fraction of the dredged material includes concentration by volume, density, fall velocity, void ratio, bulk density on the bottom, expected degree of consolidation, and grain-size distribution.

102. Disposal-mound configuration. Spatial extent and geometry of the disposal mound are important for estimating total sediment transport from unit width evaluations.

103. Disposal procedures. The disposal-operations data include the location of the barge or dredge over the disposal site during the offloading operations, as well as the period of time during the tidal cycle and days of the month when the actual disposal will occur.

104. Threshold of sediment movement. When knowledge of the characteristics of the material being placed in the disposal area is available, the resistance to movement of such material by the available ambient stresses of current and wave climate can be ascertained. Many empirically based formulas have been developed to study material movement. Several are presented herein.

105. Disposal-mound erosion rates and armoring. Prolonged scouring with only mild fluctuations (average conditions) can gradually coarsen bed sediment, leading to armoring of the bed. The depth of scour that occurs before an armor layer is fully developed can be calculated when the disposal mound steady-state erosion rates are known.

106. Effects of disposal mounding on ambient currents. As the elevation of dredged material disposal site grows with time, the ambient current condition will be deflected laterally around the sides of the mound, and flows over the top of the mound will be accelerated with corresponding pressure

changes. These changes are significant terms of sediment transport only when the depth or cross-sectional area change is large with respect to predisposal conditions.

Disposal-mound dynamics

107. The procedure of George and Walton (1984) for estimating the resuspension potential of the deposited dredged material mound involves a comparison of the bottom velocity with the threshold velocity for the known grain-size distribution. Other analyses have used the bottom friction velocity, u_* , and the wave friction factor, f_w , concept. The total maximum bottom velocity is estimated as the summation of the mean net bottom drift U_{bm} , the maximum tidal velocity, U_t , and the maximum wave-induced bottom current U_o :

$$U_{tot} = U_{bm} + U_t + U_o \quad (31)$$

where the linear concept of combining energy levels of waves and currents is considered appropriate. The maximum wave-induced bottom current can be estimated from linear wave theory as:

$$U_o = \frac{\frac{\pi H}{T}}{\sinh\left(\frac{2\pi d}{L}\right)} \quad (32)$$

where

H = wave height, ft

T = wave period, sec

d = water depth, ft

L = wave length, ft

108. The wave length depends on the wave period and the water depth; therefore, the computation of the wave length is not straightforward, but can be determined by standard coastal engineering procedures. By knowing the typical grain diameters, a threshold velocity, V_t , can be obtained from Shields (or other data sources), above which incipient motion of disposed material can be expected. If $U_{tot} > V_t$, the site will tend to be dispersive; if $U_{tot} < V_t$, the site will tend to retain the disposed dredged material. Next, the probability of resuspension can be calculated by summing all

the percentage occurrences from the wave frequency-of-occurrence table for each combination of wave height and wave period that will cause material movement in water of that depth at the disposal site.

109. Applied Science Associates, Inc. (1983), developed relationships for a volumetric sediment-transport rate on a unit width basis as a function of the mean net bottom drift velocity, U_{bm} , and various wave-induced bottom currents, U_o . A sample of these data displays is presented in Figure 15. By applying the data of Applied Science Associates, Inc. (1983), for the appropriate material (sand, silt, clay, etc.), the annual unit sediment-transport rate per meter of site width, Q_s , for each material constituent considered can be determined from the appropriate figure. The annual sediment transport rate (in cubic metres), Q_t , is the annual unit sediment transport rate, Q_s , multiplied by the width of the receiving impact area:

$$Q_t = Q_s B \quad (33)$$

Here the spatial extent of the disposal mound configuration is required for estimating the total sediment transport from these unit width evaluations.

110. If the annual sediment transport rate exceeds the annual disposal rate, no long-term mounding of the disposal site will be expected. If the annual disposal rate exceeds the annual sediment-transport rate, long-term mounding can be expected.

111. It should be noted that there are several other methods for computing sand and clay-silt transport. For example, there are the DuBoys, Ackers-White, Einstein, and other formulas for sand transport. Likewise, there are several methods for computing clay-silt resuspension rates, such as the Ariathurai equation. All of these methods are not presented here, but the Ackers-White and the Ariathurai formulas are used in examples that follow.

Example steady-state analytical procedure

112. A steady-state analytical method for mound resuspension was developed by Trawle and Johnson (1986) in an investigation of the Alcatraz (San Francisco Bay) aquatic disposal site. The objective of that investigation was to quantitatively estimate the capability of the Alcatraz disposal site to disperse dredged material that was barge-dumped at the mound location.

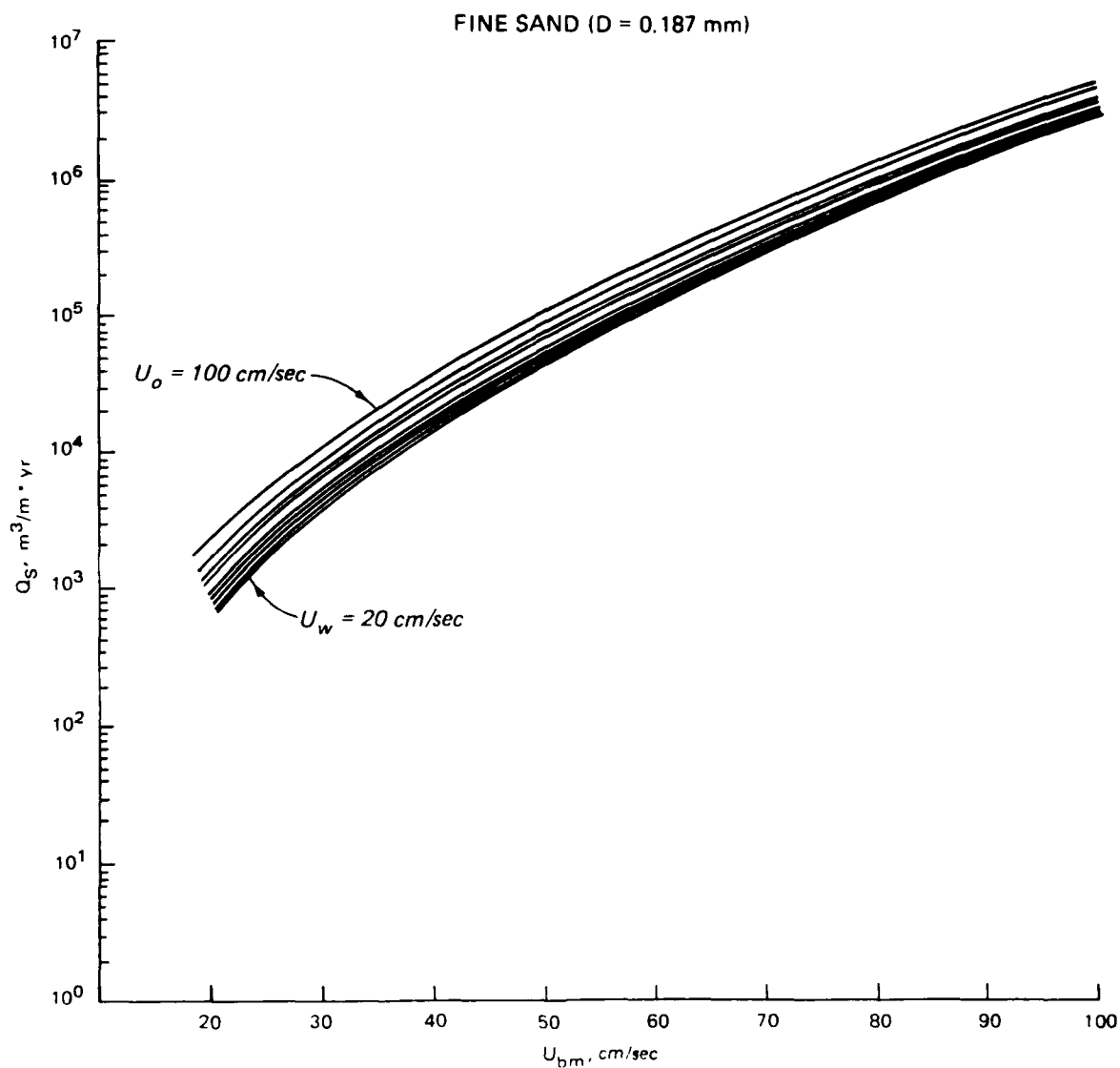


Figure 15. Volumetric sediment transport rate, Q_s , versus mean net bottom drift velocity, U_{bm} , for various wave-induced bottom currents U_o (after Applied Science Associates, Inc. 1983)

Specifically, the objective was to estimate both the percent of dumped material initially deposited at the dump site and the percent of deposited material subsequently resuspended and transported away from the dump site under varying hydrodynamic conditions. The investigation did not include the long-term fate of dumped material that leaves the disposal site.

113. The approach by Trawle and Johnson (1986) was to first simulate the barge dumping of dredged material using the computer dump model DIFID (Disposal from Instantaneous Dump). This model predicted the portion of the dumped material that would be transported from the disposal site by ambient currents before striking the Bay bottom and the portion that would be deposited within the disposal site. To estimate the amount and rate at which the deposited material would be resuspended and transported from the disposal site, an analytical approach was used. The analytical procedure included use of the Ackers-White transport function for sand transport and the Ariathurai equation for the erosion and transport of clay and silt. Appropriate values, based on type of material being dumped, for the critical shear stress for erosion and the erosion rate constant were used in the Ariathurai equation. This equation was also used to estimate erosion of consolidated clay-silt clumps or clods, again using appropriate values for the critical erosional shear stress and erosion rate constant.

114. Ackers-White sand transport model. In the development of the Ackers-White (1973) formulation, a coarse sediment is considered to be transported mainly as a bed process, and a fine sediment is considered to be transported within the body of the flow. Sediment mobility is described by the ratio of the appropriate shear force on unit area of the bed to the immersed weight of a layer of grains. The mobility number is denoted F_{gr} and is defined as

$$F_{gr} = \frac{u_*^n}{\sqrt{gD(S-1)}} \left(\frac{V}{\sqrt{32} \log \frac{\alpha d}{D}} \right)^{1-n} \quad (34)$$

where

u_* = shear velocity

n = transition exponent depending on the sediment size

g = acceleration due to gravity

D = sediment diameter
 S = mass density of sediment
 V = mean velocity of flow
 α = coefficient in rough turbulent flow
 d = mean depth of flow

115. A nondimensional sediment grain size is defined as

$$D_{gr} = D \left[\frac{g(S - 1)}{v^2} \right]^{1/3} \quad (35)$$

where v = kinematic viscosity of water. When the value of D_{gr} has been derived, the value of n , the transport exponent, can be determined as follows:

$$\begin{aligned}
 \text{for } D_{gr} &\leq 1.0, & n &= 1 \\
 \text{for } 1.0 < D_{gr} &\leq 60, & n &= 1 - 0.56 \log D_{gr} \\
 \text{for } D_{gr} &> 60, & n &= 0
 \end{aligned}$$

and the value of the sediment mobility number, F_{gr} , can be calculated from Equation 34.

116. The Ackers-White approach uses dimensionless expressions for sediment transport based on the stream power concept, in the case of coarse sediment using the product of net grain shear and stream velocity as the power per unit area of bed, and for fine sediment using the total stream power. The dimensionless sediment transport rate, G_{gr} , is described by the expression

$$G_{gr} = C \left(\frac{F_{gr}}{A} - 1 \right)^m \quad (36)$$

where

C = coefficient in sediment transport function
 A = value of F_{gr} at nominal initial motion
 m = exponent in sediment transport function

The values of C , A , and m can be derived as follows:

$$\begin{aligned}
 \text{for } 1 < D_{gr} &\leq 60, & C &= 2.86 \log D_{gr} - (\log D_{gr})^2 - 3.53 \\
 & & A &= (0.23 / \sqrt{D_{gr}}) + 0.14 \\
 & & m &= (9.66 / D_{gr}) + 1.34
 \end{aligned}$$

for $D_{gr} > 60$,

$$C = 0.025$$

$$A = 0.17$$

$$m = 1.50$$

117. When the dimensionless sediment-transport rate has been derived from Equation 36, the sediment transport in mass flux per unit mass flow rate, X , can be determined from the expression

$$X = \frac{G_{gr}^{SD}}{d} \left(\frac{v}{u_*} \right)^n \quad (37)$$

118. According to Trawle and Johnson (1986), typical representative steady-state sample computations may involve quantities such as:

D = representative grain diameter = 0.0002 m

S = density of sediment = 2.60

ν = kinematic viscosity of water = 0.000001 m²/sec

g = acceleration due to gravity = 9.81 m/sec²

Substituting in Equation 35 yields

$$D_{gr} = 4.96$$

119. Substituting other typical values for Equation 34

d = mean depth of flow = 12.19 m

α = coefficient in rough turbulent flow = 12.3

u_* = shear velocity from Manning's shear stress expression
= $(\sqrt{g} NV)/d^{1/6}$

n = transition exponent based on sediment size
= $1.0 - 0.56 \log D_{gr} = 0.61$

N = Manning's friction factor = 0.015

V = tidal current, m/sec

yields

$$F_{gr} = 0.05472V$$

120. For Equations 36 and 37 with

C = coefficient for sediment transport function = 0.00943

A = value of F_{gr} at nominal initial motion = 0.234

m = exponent in sediment transport function = 3.29

$$G_{gr} = 0.00943 \left(\frac{0.5472V}{0.243} - 1 \right)^{3.29}$$

$$X = 0.00138 G_{gr} V^{-0.39}$$

121. The mass flow rate per unit width, q , is defined as:

$$q = \gamma_w Vd \quad (38)$$

where

γ_w = unit weight of water (1,000 kg/m³)

q = 12,190 (V), kg/sec/m

122. The sand transport by weight per unit width, T , is given as:

$$T = Xq \quad (39)$$

or

$$T = 16.82 G_{gr} V^{-0.39}, \text{ kg/sec/m}$$

The functional curve relating the sand-transport capability at the disposal mound (in pounds/minute/foot) to tidal current (in feet/second) is shown in Figure 16. As shown, tidal currents of about 3 fps are required for sand movement to begin.

123. Ariathurai clay and silt transport model. Quantification of erosion rates of silt-clay sediment is difficult in view of the many variables involved, such as the chemical characteristics of the material, the degree of consolidation, armoring, and the physical and chemical properties of the water. An equation based on work by Parthenaides (1962) was developed by Ariathurai and Arulanadan (1978) and used by Trawle and Johnson (1986) to

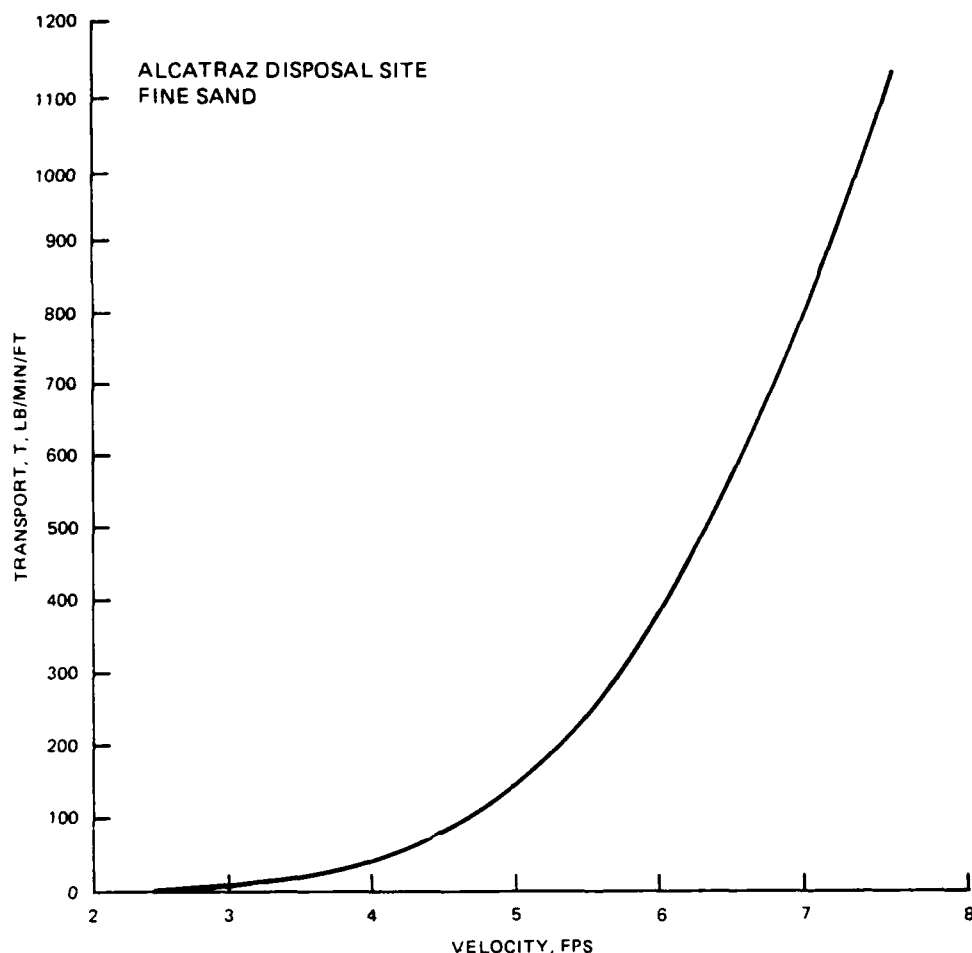


Figure 16. Transport potential of sand at disposal mound as a function of current speed, based on Ackers-White method (after Trawle and Johnson 1986)

estimate the subsequent erosion and resuspension of clay and silt which settled to the bottom. The same equation with different coefficients was also used to estimate erosion of both relatively unconsolidated clay-silt and clumps of consolidated clay-silt which result from clamshell dredging operations.

124. Small-scale laboratory experiments indicate that partially consolidated cohesive material is eroded in direct proportion to applied shear stresses, and that the process is independent of suspended load concentration. In equation form, this relationship, referred to as the Ariathurai equation, is:

$$S = M \left[\frac{\tau_B}{\tau_c} - 1 \right] \quad (40)$$

where

S = erosion rate

M = erosion rate constant

= 0.002 kg/sec/m² for unconsolidated material

= 0.0005 kg/sec/m² for clumps

τ_B = bed shear stress

τ_c = critical shear stress for erosion

and

$$\tau_B = \frac{\rho g n^2 V^2}{d^{1/3}} \quad (41)$$

where

ρ = fluid density

g = acceleration due to gravity

n = Manning's friction factor

V = tidal current

d = water depth

125. Typical representative steady-state sample computations may involve quantities such as:

M = erosion rate constant = 0.002 kg/sec/m² for unconsolidated material

M = erosion rate constant = 0.0005 kg/sec/m² for clumps

ρ = fluid density = 1,000 kg/m³

g = acceleration due to gravity = 9.81 m/sec²

n = Manning's friction factor = 0.015

V = tidal current, m/sec

d = water depth = 12.19 m

τ_c = critical shear stress for erosion = 0.10 N/m² for
unconsolidated material

τ_c = critical shear stress for erosion = 1.00 N/m² for clumps

These values give

$$S_u = 0.0194 V^2 - 0.002, \text{ kg/sec/m}^2 \quad \text{for unconsolidated material}$$

$$S_c = 0.000485 V^2 - 0.0005, \text{ kg/sec/m}^2 \quad \text{for clumps}$$

The functional curve for the above values relating clay-silt resuspension capability at the disposal mound (in pounds per minute per square foot) to tidal current (in feet per second) is shown in Figure 17. The results for both consolidated and unconsolidated materials are presented in this figure.

Time- and Rate-Dependent Analytical Methods

126. Turbulent flow and wave motions are, in the strictest sense, unsteady; however, if average values are taken over sufficiently long periods of time, the resulting motions and accelerations may be evaluated on a steady-state basis. The rise and fall of the tide creates a tidal current that varies both in direction and in magnitude. For certain physical processes, the tidal velocity may be characterized by one value, and the phenomenon becomes quasi-steady, allowing for a significant amount of information to be gained with a minimum of analysis effort being expended in the process. When it is determined that more detailed information is required than that available through the averaging process, it becomes necessary to treat the time- and rate-dependent physical processes in an unsteady manner.

Discretization and sequencing of events

127. The simplest approach to the estimation of the resuspension potential from a dredged material disposal mound is to select some representative average condition for each of the energy sources and disposal operations. These averages are then combined into a single analysis of the resuspension and erodibility of the mound for that composite representative condition. Such a composite condition can be developed for several finite conditions, including storm events.

128. The selection of these representative levels of energy sources must be made with an understanding of the impact of these averages. Because many of the relationships between energy sources and sediment movement are non-linear, the selection of an average condition must be made with the non-linearity in mind. For example, the movement of sediment due to the wave energy is normally related to energy flux, which is accordingly proportional to the square of the wave height. Thus, the greatest rate of sediment transport occurs at the larger wave heights. Therefore, a straightforward selection of a single representative average wave height (or other energy

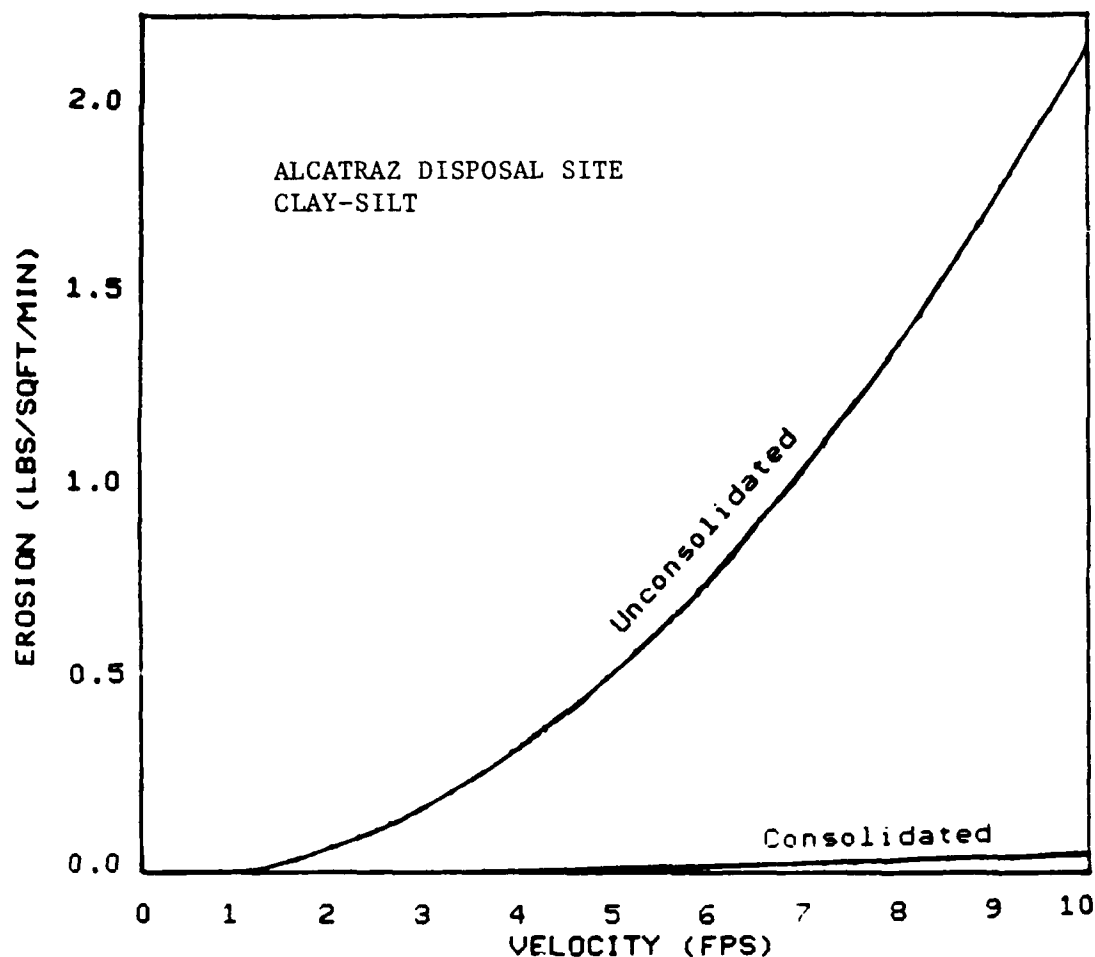


Figure 17. Erosion potential of clay-silt particles at disposal mound (unconsolidated and consolidated) as a function of current speed, based on the Ariathurai equation (after Trawle and Johnson 1986)

source) may not provide the degree of insight required to evaluate the resuspension of dredged material from the disposal site.

129. Most of the physical phenomena associated with sediment movement and energy sources that cause such movement are continuous. That is, they are essentially continuous processes that vary smoothly with time and position. The computing systems designed to process numerical simulations of these physical processes are digital and discrete. Hence, it becomes necessary to digitize (discretize) the continuous process into a series of distinct elements, to select those discrete values of the various energy sources at the appropriate time, and to simulate the processes as a definite series of time events or unit steps of the rate changes that may be occurring. Such digitization of

continuous data is readily applicable to tidal elevations or tidal current values at a location. Here multiples of 1-hr increments can be utilized to cover several tidal cycles, or even several days or weeks.

Example time- and rate-dependent analytical procedure

130. The steady-state analytical method for evaluating mound resuspension at the Alcatraz aquatic disposal site, developed by Trawle and Johnson (1986), was extended to cover time- and rate-dependent conditions as well. This analysis was a time-increment application of the Ackers-White (1973) for sands and the Ariathurai (1977) for clays and silts transport formulas that estimated the capability of the ambient currents to remove material from the disposal site over a tidal cycle.

131. Disposal site currents were collected in the San Francisco Bay-Delta physical hydraulic model by Tetra Tech, Inc. (1984), for five different hydrodynamic conditions used in the Trawle and Johnson (1986) analytical analysis. The five conditions are as follows:

<u>Series</u>	<u>Tide Range</u>	<u>Delta Net Outflow cfs</u>
11	19-year mean	4,400
12	19-year mean	40,000
21	Neap	4,400
22	Neap	40,000
31	Spring	4,400

132. The physical model testing included both the bathymetric condition that existed at the disposal site prior to the depth loss and the bathymetric condition recently observed in which a mound has developed at the disposal site, resulting in significant loss of depth. The mound-out condition had a depth of 160 ft at the disposal site, while the mound-in condition had a depth of only 29 ft. The mound-in currents from the physical model study (Figure 18) were used as input to DIFID in this effort since the bottom contours are the same except that the mound peak was excavated to 40 ft for the DIFID runs.

133. In order to evaluate the sensitivity of the site's dispersive capability to the current environment, another current condition was tested.

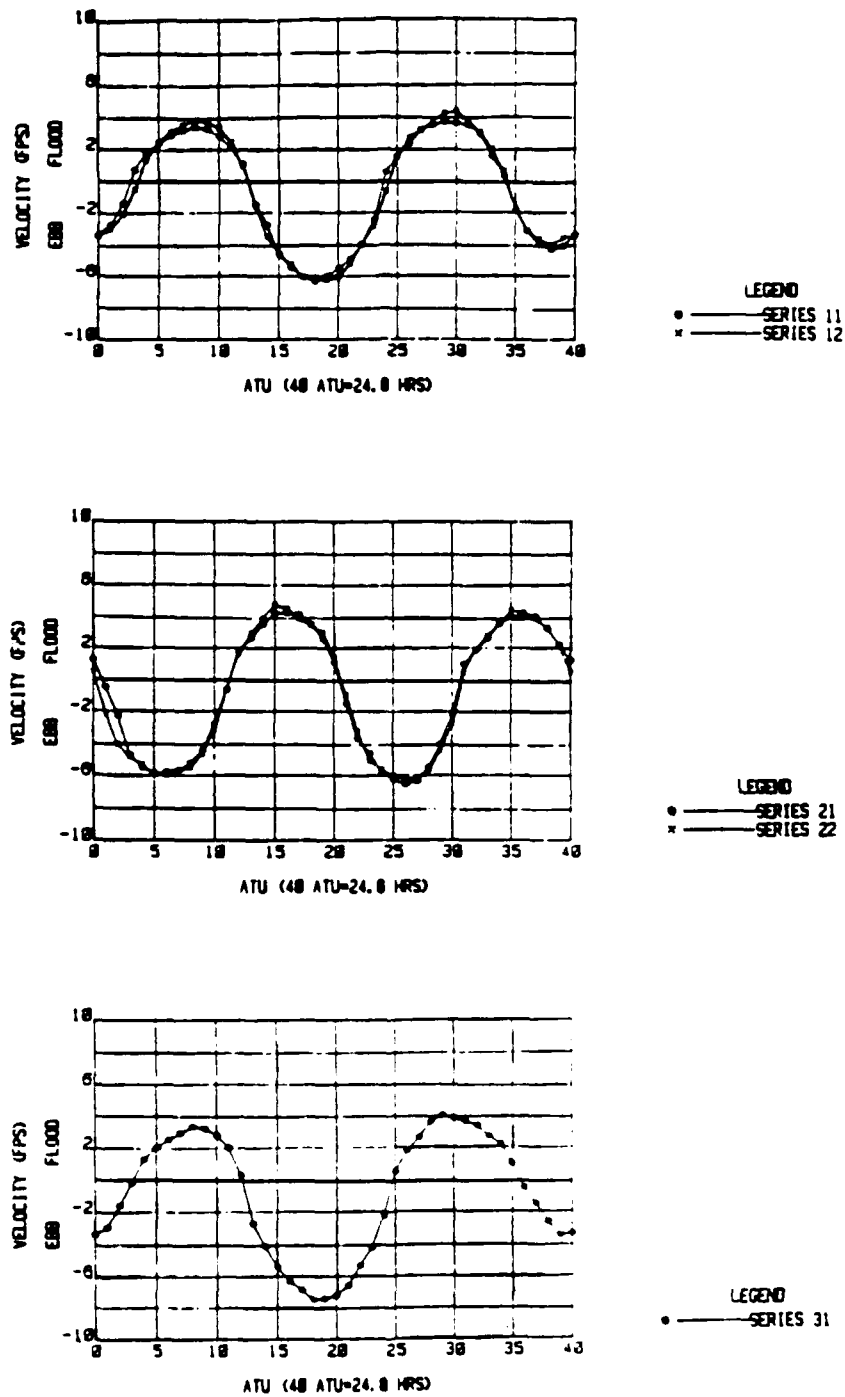


Figure 18. Near-bottom velocities measured at the mound in the San Francisco Bay-Delta model for the Alcatraz disposal site evaluation (after Trawle and Johnson 1986)

This additional test, referred to as Series XX, was simply the currents from Series 11 multiplied by an arbitrarily selected two-thirds factor. Series XX had maximum near-surface ebb currents of 4.2 fps and maximum flood currents of 2.9 fps.

134. The material simulated in the barge dump consisted of 60-percent clay-silt particles ranging in size from 0.02 to less than 0.002 mm and 40-percent fine sand ranging in size from 0.2 to 0.06 mm (Trawle and Johnson 1986). The bulk density of the barge slurry was 1.44 g/cm³, resulting in a moisture content for the slurry of about 74 percent. Testing included a slurry with no clumps to simulate barge material obtained from a hydraulic dredging operation and a slurry in which 30 percent of the clay-silt fraction was in the form of clumps or clods to simulate barge material from a clamshell or bucket dredging operation. The disposed material characteristics are shown in Table 1.

Table 1
Characterization of the Dredged Material
Alcatraz Dredged Material Disposal Site
San Francisco Bay, California

	<u>No Clumps</u>	<u>30% Clumps</u>
Sand concentration by volume, ft ³ /ft ³	0.1056	0.1056
Silt-clay concentration by vol, ft ³ /ft ³	0.1584	0.1109
Clumps concentration by vol, ft ³ /ft ³	--	0.0475
Sand density, g/cm ³	2.60	2.60
Silt-clay density, g/cm ³	2.60	2.60
Clump density, g/cm ³	--	2.60
Fluid density, g/cm ³	1.018	1.018
Sand fall velocity, fps	0.026	0.026
Clump fall Velocity, fps	0.50	0.50
Sand voids ratio	0.80	0.80
Silt-clay void ratio	0.80	0.80
Clump void ratio	--	0.90
Bulk density, g/cm ³	1.436	1.436
Aggregate voids ratio	0.80	0.80

135. Based on the Ackers-White (1973) transport formula for the sand fraction of the dumped material, the transport potential on the mound in 40 ft of water for each condition is given in Table 2 in cubic feet per tidal cycle per foot of width. The incremental computations are similar to that of the steady-state conditions where averaged values were utilized. Since the width of the disposal mound transverse to flow in 40 ft of water was estimated to be about 400 ft, the transport potentials on the mound for Series 11, 12, 22, 31, and 32 are 36, 42, 68, 68, and 90 million pounds of sand per tidal cycle, respectively. Transport potential for Series XX was reduced to 3 million pounds of sand per tidal cycle. Time-dependent movement of sand over a tidal cycle for Series 22 conditions is given in Table 3.

Table 2
Sand Transport Potential Alcatraz Dredged
Material Disposal Site San Francisco Bay, California

<u>Series</u>	<u>Transport Potential lb/tidal cycle/ft width</u>
11	93,200
12	107,400
21	166,500
22	168,400
31	224,500
XX	9,800

136. Based on the Ariathurai equation for the clay-silt fraction of the dumped material, the erosion rate potential on the mound in 40 ft of water for each condition for both unconsolidated and consolidated material is given in Table 4 in cubic feet per tidal cycle per square foot of bottom. The computations are similar to the steady-state conditions when averaged values were evaluated.

137. Since the bottom area of the mound in 40 ft of water was estimated to be about 160,000 sq ft, the erosion potentials on the mound for Series 11, 12, 21, 22, and 31 are 58, 62, 79, 80, and 74 million pounds of clay-silt per

Table 3

Transport Potential for Sand Based on Ackers-White MethodSeries 22, Alcatraz Dredged Material Disposal SiteSan Francisco Bay, California

<u>ATU*</u>	<u>Velocity, fps</u>	<u>Transport (T), lb/min/ft</u>
0	0.5	0
1	2.2	1
2	4.1	52
3	4.9	137
4	5.3	210
5	5.9	359
6	5.8	330
7	5.7	303
8	5.5	254
9	4.7	110
10	3.2	13
11	0.7	0
12	1.6	0
13	2.9	6
14	3.9	38
15	4.7	110
16	4.5	87
17	3.9	38
18	3.4	17
19	3.0	7
20	1.4	0
21	1.0	0
22	3.3	14
23	5.1	168
24	5.6	278
25	6.1	423
26	6.2	458
27	6.2	458
28	5.7	303
29	4.5	87
30	2.8	4
31	0.5	0
32	2.0	0
33	2.8	4
34	3.6	24
35	4.4	77
36	4.3	68
37	3.9	38
38	3.3	14
39	2.2	1
TOTAL		168,400 lb/tidal cycle/ft

* ATU = Acquisition time unit, 40 ATU = 1 tidal cycle (25 hr)
(1 ATU = 37.5 min).

Table 4
Clay-Silt Erosion Potential
Alcatraz Dredged Material Disposal Site
San Francisco Bay, California

<u>Series</u>	<u>Erosion Potential</u> <u>lb/tidal cycle/sq ft</u>	
	<u>Unconsolidated</u>	<u>Clumps</u>
11	363	3.4
12	390	4.2
21	497	7.2
22	508	8.0
31	462	6.8
XX	140	0.4

tidal cycle, respectively, for the unconsolidated clay-silt, and 0.5, 0.7, 1.2, 1.3, and 1.1 million pounds of clay-silt per tidal cycle, respectively, for the clumps. For Series XX the transport potential for unconsolidated clay-silt was 22 million pounds per tidal cycle and for consolidated clay-silt only about 0.1 million pounds per tidal cycle. Tabulated results of clay-silt resuspension summed over a tidal cycle (in pounds per square foot) for Series 22 are presented in Table 5.

Modeling

Physical Modeling

138. Movable-bed. Physical models of coastal and estuarine bed material movement are three-dimensional investigations that involve littoral transport, onshore-offshore transport, possibly scour or erosion around structures, and erosion or deposition at offshore areas of interest (sand-mining source, open-water dredged material disposal site, etc.). A movable-bed model study is required to understand the effects of coastal or offshore modifications on such transport phenomena, and of the transport of such movable material on the functional operation of other coastal engineering structures.

Table 5
Resuspension Potential for Clay-Silt Based on
Ariathura⁴ Equation, Series 22
Alcatraz Dredged Material Disposal Site
San Francisco Bay, California

<u>ATU</u>	<u>Velocity, fps</u>	<u>S_u, lb/sq ft/min</u>	<u>S_c, lb/sq ft/min</u>
0	0.5	0.00	0.00
1	2.2	0.08	0.00
2	4.1	0.35	0.00
3	4.9	0.50	0.01
4	5.3	0.59	0.01
5	5.9	0.73	0.02
6	5.8	0.71	0.01
7	5.7	0.68	0.01
8	5.5	0.64	0.01
9	4.7	0.20	0.00
10	3.2	0.20	0.00
11	0.7	0.00	0.00
12	1.6	0.03	0.00
13	2.9	0.16	0.00
14	3.9	0.31	0.00
15	4.7	0.46	0.00
16	4.5	0.42	0.01
17	3.9	0.31	0.00
18	3.4	0.23	0.00
19	3.0	0.17	0.00
20	1.4	0.02	0.00
21	1.0	0.00	0.00
22	3.3	0.22	0.00
23	5.1	0.54	0.01
24	5.6	0.66	0.01
25	6.1	0.80	0.02
26	6.2	0.82	0.02
27	6.2	0.82	0.02
28	5.7	0.68	0.01
29	4.5	0.42	0.01
30	2.8	0.15	0.00
31	0.5	0.00	0.00
32	2.0	0.06	0.00
33	2.8	0.15	0.00
34	3.6	0.26	0.00
35	4.4	0.41	0.01
36	4.3	0.38	0.01
37	3.9	0.31	0.00
38	3.3	0.21	0.00
39	2.2	0.08	0.00
Total		508 lb/sq ft/tid cycle	8.0 lb/sq ft/tid cycle

139. A completely quantitative movable-bed model investigation of coastal and estuarine erosion appears to be impractical within the present state of the art. However, movable-bed, scale-model investigations of several types of coastal erosion problems are feasible and can be conducted in such a way that useful, and sufficiently accurate, information can be obtained for design purposes. If adequate prototype data are available, and verification procedures in the model are successful, an investigator should have confidence in the results of coastal movable-bed models.

140. A satisfactory movable-bed, scale-model investigation is perhaps the most difficult type of physical model study to perform. There is a transition from one basic regime of boundary flow to another as sediment motion outside the surf zone is compared with sediment motion within the surf zone; thus, exact dynamic similitude of the dominant physical processes in both regimes simultaneously (using the same model fluid and the same model laws for reproducing the waves, currents, and bottom material for both regimes) is impossible. Other effects, such as edge waves, rip currents, the directional wave spectrum, the long-wave environment, and the prototype sediment-size distribution and amount of sorting, must all be considered at least to the point of showing that they are unimportant for the particular movable-bed study under investigation.

141. Additional applied research on coastal movable-bed modeling should be performed to further the science. However, this does not mean that sufficiently accurate model studies cannot now be performed, but rather it indicates that comprehensive planning and serious thought are required to conduct such studies. More importantly, it means that the conduct of a movable-bed physical hydraulic model study should not be rushed. Adequate time and research funds must be allocated to ensure that accurate model results are obtained. Additional discussion on the theoretical and practical aspects of movable-bed physical modeling is presented in Appendix B.

142. Galveston disposal site physical model study. Apparently, the only completely three-dimensional physical modeling of the movable aspects of dredged material disposal in the open ocean was conducted by Simmons and Boland (1969). That study was performed to investigate the possibility of disposed dredged material reentering at the navigation channel entrance to Galveston Harbor. The movable model material was scaled-dimension crushed coal that was subjected to model testing procedures which forced the crushed

coal movable bed in the model to respond in geometry as the prototype sand-silt-clay mixtures had evolved. This is a trial-and-error verification procedure that answers most questions regarding shoaling and erosion of navigation channels, but may not entirely satisfy all relationships involved with open-ocean dredged material disposal site dynamics.

143. Verification of the Galveston Harbor model was accomplished in two separate but interrelated phases. Phase 1 was the hydraulic verification that resulted in proper reproduction of prototype tidal heights, tidal phases, current velocities, and current directions throughout the entire model. Phase 2 consisted of verification of bed movement and resulted in an accurate reproduction of observed trends of prototype bed movement, as illustrated by comparison of successive hydrographic surveys of the problem area, and which also permitted empirical determination of the model time scale for bed movement. Two separate series of tests were performed in the moveable-bed model to evaluate the dispersion of material from the open ocean dredged material disposal site. This existing dredged material disposal site, used for hopper dredging activities in Galveston Harbor entrance, is located about 3 miles off the end of the south jetty.

144. The test results (Figure 19) showed the prevailing directions of movement of bed materials from the disposal area for the different wave directions tested. The test data indicated that wave directions south of about S 45° E cause movement of bottom sediments toward and into the outer bar channel of the entrance channel, while movement caused by wave directions east of about S 45° E was generally to the south of the entrance channel. The test results also indicated that most of the material that moved toward or into the outer bar channel originated in the northeasterly one-third of the existing disposal area, while the direction of material movement from the remaining two-thirds was generally to the south of the entrance channel.

145. Tests were conducted in the movable-bed model for the purpose of quantifying the movement of dredged material placed in the existing open-ocean disposal site. The tests were conducted for a total of 12 years prototype time to allow sufficient time for tracing the patterns of material movement from the disposal site. Figure 20 shows that the progressive movement of material from the disposal area was largely to the east beach area, south of the south jetty. These are scour and fill contours of material movement since the initiation of the 12-year testing period, and are not bottom topography

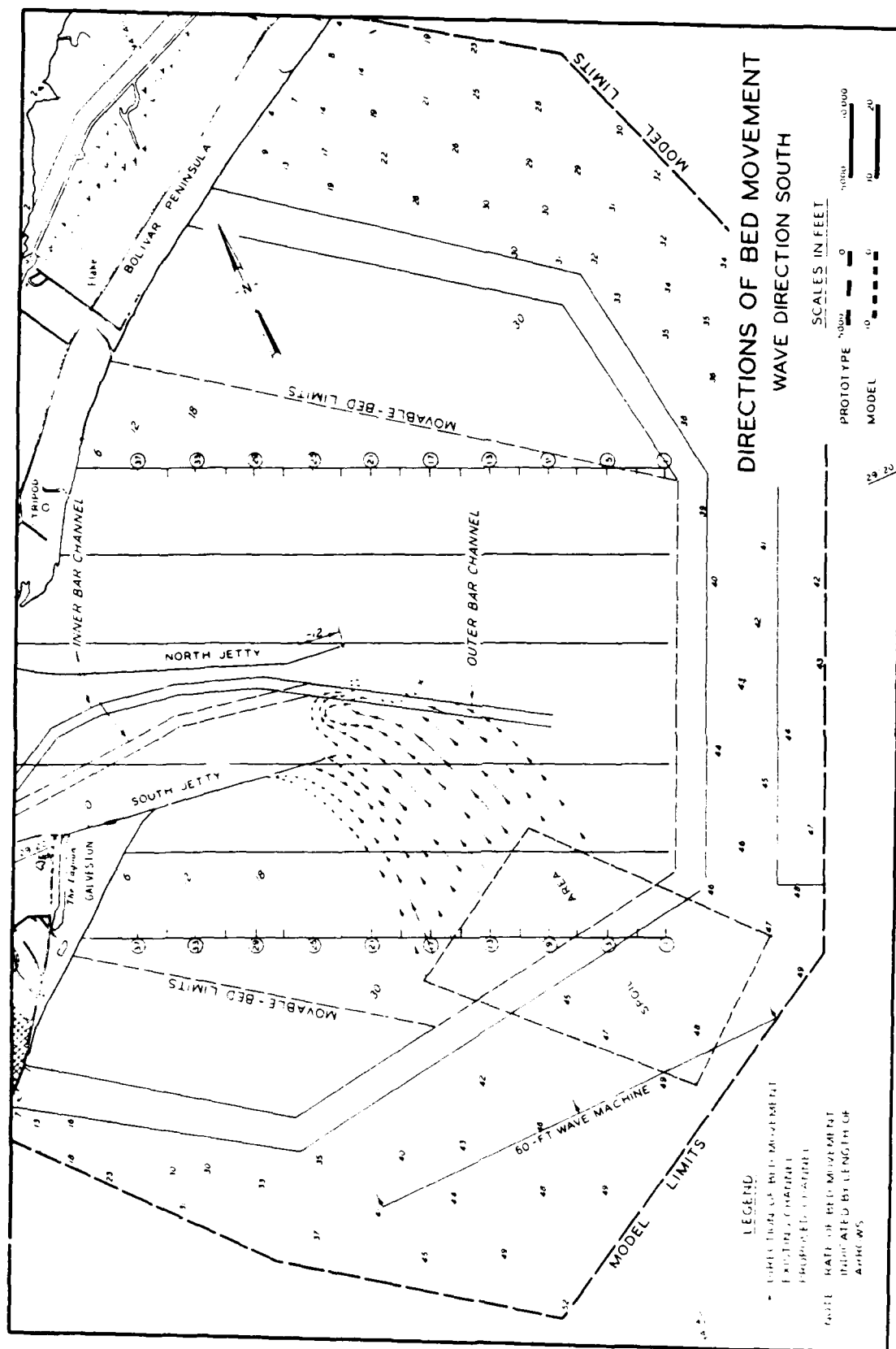


Figure 19. Qualitative indication of material movement from open-ocean dredged material disposal site, Galveston, Tex. (after Simmons and Boland 1969)

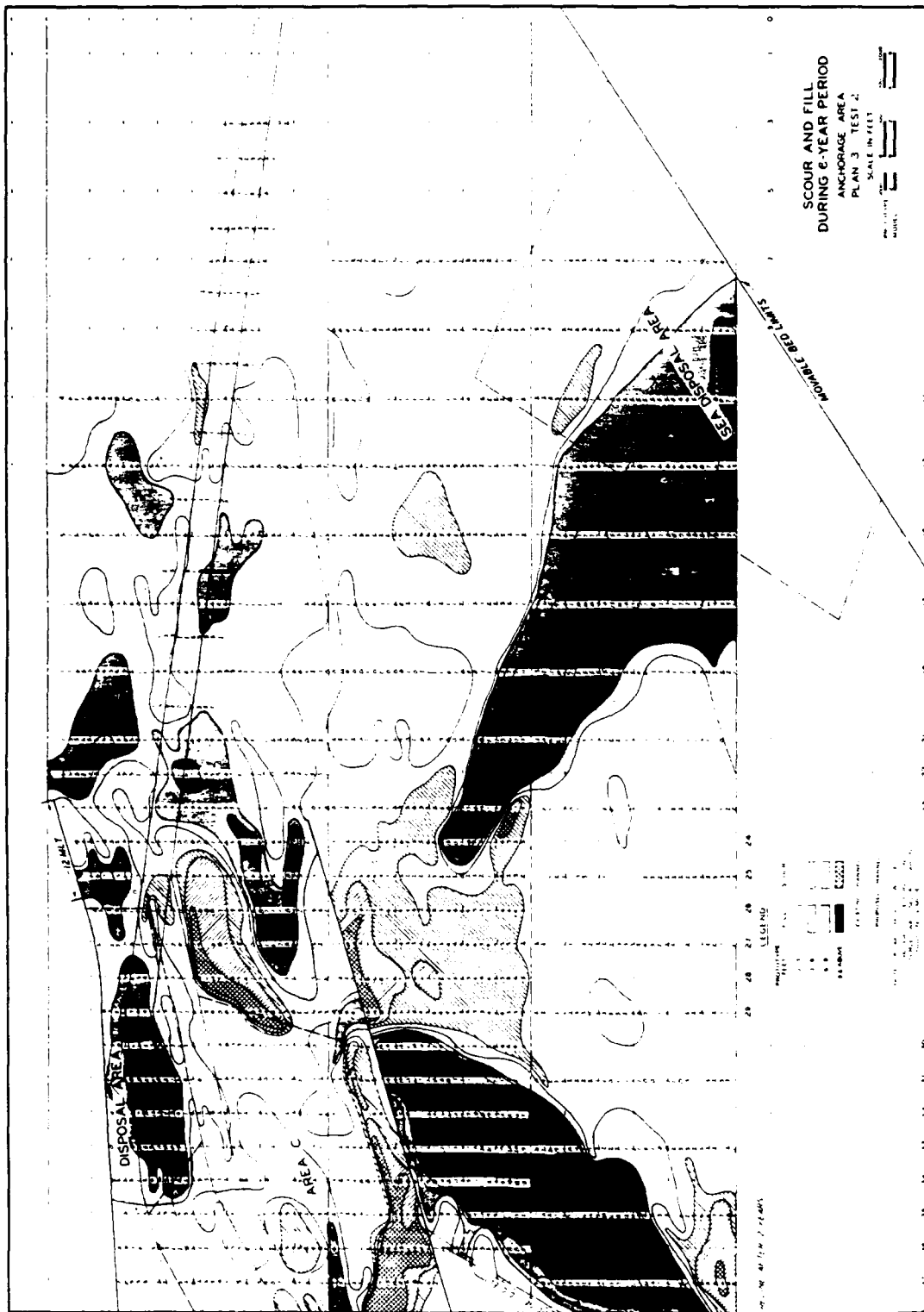


Figure 20. Scour and fill after 6 years, open-ocean dredged material disposal site, Galveston, Tex. (after Simmons and Boland 1969)

contours. Data obtained from these tests indicated that material placed in the existing open-ocean disposal site will move primarily in the direction of the waves, which in these tests was toward N 37° W (from S 37° E). It appeared from the results of these tests that material placed in the northeasterly one-third of the disposal area, 3 miles off the outer end of the south jetty, will return to the entrance channel; dredged material placed in the remaining two-thirds moves generally to the east beach area south of the south jetty and will not return to the channel.

Numerical modeling

146. Numerical simulation of erosion, transport, and redeposition of sediments from DMADS requires application of hydrodynamic models and sediment transport models that include the appropriate processes. Both types of numerical models require input data, which may be broadly classified as:

- a. Initial conditions: Data that describe the initial state of the system prior to numerical modeling.
- b. Boundary conditions: Data that specify the system geometry and the value of the calculated variables at the boundaries of the computational area.
- c. Verification requirements: Any other data necessary for verification of the numerical models.

147. A numerical model study can never be more accurate than the information on which it is based; hence, the importance of adequate field data cannot be overemphasized. The first steps in any numerical model study must be the specification of objectives; an assessment of the geophysical, chemical, and biological factors involved; and collection of data essential to describe these factors. Assessment and data collection must include:

- a. Identification of freshwater inflow sources, including their average, range, and time-history distribution of such inflow.
- b. Assessment of the tides and tidal currents that exist within the region of interest.
- c. Evaluation of wind effects and other geophysical phenomena that may be peculiar to the specific study.
- d. Understanding of wave climate throughout the region of interest, including seasonal and annual distribution with frequencies of occurrence by height, period, and direction of approach.
- e. Knowledge of the resulting wave orbital currents.

- f. Evaluation of the effects of simultaneous occurrences of unidirectional flow (tidal currents or freshwater river inflow) and oscillatory currents (wave orbital particle motion).
- g. Assessment of effects and probability of occurrence of aperiodic extreme meteorological events such as storms.
- h. Identification of types of sediments being deposited in a DMADS, and the deposition frequency of occurrence with respect to the frequency of occurrence of the stresses available for material movement.
- i. Archive of available hydrographic, bathymetric, topographic, or other geometric data pertinent to the preparation of the numerical model.

148. A preliminary assessment of pertinent and available data is necessary to provide a basis for the selection of the numerical models needed, and to provide a basis for planning field data acquisition programs. The most satisfactory procedure is to plan the numerical modeling and field data acquisition program simultaneously. If possible, the basic hydrodynamic model should be operational during the period in which the field data are being acquired. One major reason for concurrent model simulation and data acquisition is that anomalies in field data frequently occur, and the numerical model may be useful in identifying and resolving any such anomalies.

149. Numerical models of hydrodynamic processes and sediment transport are said to be coupled if they are applied simultaneously and interactively on a computer system and the sediment calculations influence the hydrodynamic calculations. If, conversely, the hydrodynamic model is run and the output from it is used as input to the sediment transport model, the two models are said to be uncoupled. In many instances it is more economical to run uncoupled models.

150. The various numerical models may be classified as one-, two-, or three-dimensional. The one-dimensional models treat the system by averaging over a succession of cross sections. One-dimensional models are well suited to geometric situations such as channels with relatively uniform cross-sectional shape and with center lines whose radius of curvature is relatively large compared with the width, provided the water density is uniform over the cross section. Two-dimensional models may be either depth- or breadth-averaged. Two-dimensional depth-averaged models are the type most commonly employed and are well suited to studies in areas such as shallow estuaries

where the water column is relatively well mixed. Breadth-averaged models are used in studies of relatively deep and narrow bodies of water with significant variation of density vertically through the water column. Three-dimensional hydrodynamic models are relatively new and have been applied to only a limited number of practical studies. In general, two-dimensional model, and three-dimensional models are more complex and more expensive than one-dimensional models. Hence, in situations where it is known a priori that one of the simpler models will produce satisfactory results, the simpler model should be employed for economy. For additional discussions on numerical models, refer to Appendix C. The joint use of physical and numerical models is discussed in Appendix D.

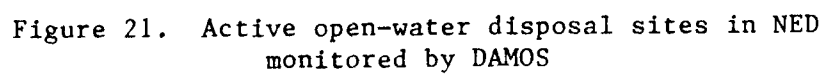
Field and Laboratory Studies

151. Some examples of field and laboratory studies of erosional characteristics of DMADS are presented in this section. For additional guidance on the use of field and laboratory studies, refer to Appendix A.

Field studies

152. New England region. The Disposal Area Monitoring System (DAMOS) program was initiated in 1977 by the US Army Engineer Division, New England (NED), to provide scientific data for management and monitoring of dredged material disposal sites throughout New England. The New England region requires dredging from a wide range of estuaries and disposal of material at a variety of disposal sites; hence, a flexible management plan capable of addressing the individual parameters of each situation is necessary. Measurements are currently made throughout the New England region at the disposal sites shown in Figure 21. In addition to monitoring of disposal operations, the basic principles of in-situ measurement are also applied to other aspects of disposal management such as site designation, control of disposal procedures, and development of management strategies such as containment, capping, burial, or point dumping.

153. Since detection of contaminants derived from dredged material is extremely difficult, monitoring of the stability of a dredged material deposit becomes correspondingly important. If material is contained within a stable mounded deposit, then the contaminants are not available to the environment and will not occur beyond the margins of the disposal site. Therefore,



one of the most important concepts that the DAMOS project has provided to disposal management is the classification of disposal points as containment or dispersal sites. This concept is then reflected in the overall management of the site. If containment of material can be achieved, then point dumping, creation of disposal mounds, capping, and continued monitoring of the impacts of disposal in the vicinity of the site are all possible. However, if the site is a dispersal site, then point dumping or other techniques to limit the spread of material are not warranted, and monitoring of impacts is much more difficult simply because of a much larger zone of potential impact. For these reasons the general policy for designation of disposal sites in New England has been to find areas where dredged material is relatively stable and contaminants can be contained within the disposal mounds created. Over the years, evidence has continually pointed to the conclusion that, with the exception of Cornfield Shoals, all sites studied under the DAMOS program can be considered as containment sites.

154. Dredged material stability has been monitored primarily through bathymetric techniques and REMOTS to evaluate changes in topography that could result from energy levels sufficient to cause erosion and transport of dredged material. The results of these surveys have shown conclusively that dredged material at all sites is stable under normal conditions. However, storm events and bioturbation can cause resuspension and transport of material on both large and small scales. Bioturbation by macrofauna appears to be a major factor in breaking down the larger clumps of material associated with clam-shell dredging, and benthic infauna can provide either a stabilizing effect or bioturbation effect, depending on the type of organism present.

155. The only major change in topography observed during DAMOS monitoring efforts occurred at the Central Long Island Sound site during Hurricane David in the fall of 1979. This resulted in a significant loss of material from the top of the silt cap, but no change in any of the older disposal mounds in the same area. Since that time, no changes in that mound have been observed, and it was concluded that the occurrence of the storm before the dredged material had stabilized was the probable reason for erosion at the site.

156. None of the disposal mounds has given any indication of bed-load transport resulting in a spread of material in the vicinity of the flanks, and it appears that if material is in suspension, it is transported a substantial

distance from the site before being deposited. Consequently, the dilution of material in background sediment loads makes detection of dredged material extremely difficult, if not impossible. In addition to bathymetric techniques for measuring stability, photography, diving observations, and sediment sampling also provide input toward assessing stability and behavior of the dredged material. All have indicated no significant loss of material once disposal is complete and consolidation of deposits is in advanced stages. Remote sensing of dredged material and the mussel watch program have also indicated no detection of dredged material since disposal has ceased.

157. With the knowledge that disposal sites designated throughout New England are containment sites, specific procedures for disposal of dredged material have been developed to create dredged material mounds. The taut-wire moored buoys have been extremely successful in Long Island Sound where tugs have stopped the disposal scows close to the buoys, but less successful in open-water areas where longer hawsers are used and the scows are moving during disposal. Depending on the volume of material to be dumped, and the requirements to create a small mound, motion of the disposal scow appears to be an important factor in point dumping and creation of mounds.

158. Columbia River, Oregon. The field investigation of the open-ocean dredged material disposal site at the mouth of the Columbia River, Oregon, was one of the Aquatic Disposal Field Investigations (ADFI) of the Dredged Material Research Program (DMRP). This site was selected as an ADFI location because it possessed all the criteria for an open-ocean field research region: (a) regulatory approval for open-ocean disposal; (b) disposal in at least 10 m of water; (c) annual disposal capability; (d) availability of experience to conduct research; and (e) regional representation. The area also was accessible for monitoring, was considered to be a top-priority ongoing dredging project with sufficient quantities of available material, and had major research facilities nearby.

159. The study area was located adjacent to the mouth of the Columbia River on the Oregon-Washington continental shelf, where water depths range between 10 and 90 m. The Columbia River plume influences an area from latitude 40° to 49° N to as far as 600 km offshore. An experimental disposal area was selected for investigation in June 1975. It was located in approximately 26 m of water and was designated special purpose. This area had not been used for disposal in the past and, hence, provided an opportunity to study disposal

effects in an undisturbed open-ocean environment. During the period 9 July 1975 to 26 August 1975, 458,632 cu m of dredged material removed from the entrance channel was disposed here. The investigation began in August 1974 as an effort to identify and evaluate the effects of open ocean disposal on the flora and fauna at the disposal site, as well as to measure rates and patterns of dispersion of dredged material following disposal. This research included evaluations of both short- and long-term effects of coastal disposal operations.

160. This disposal produced a distinct bathymetric feature generally confined to a radius of 460 m around the marker buoy, with accumulation primarily to the south and west of the buoy. Immediately following the disposal experiment, the volume of dredged material at the area was estimated to be 324,000 cu m (Boone et al. 1978), representing approximately 71 percent of the material released at the area. It was not suggested that 29 percent of the material released did not arrive at the bottom, although undoubtedly some of the silt and clay particles were carried in suspension away from the disposal area. It was suggested that certain inaccuracies exist in the volume calculations, in the bathymetric surveying, and particularly in the estimation of the amount of material actually released by the hopper dredge. Subsequent bathymetric surveys performed in February 1976 revealed that the volume of the disposal mound had decreased to approximately 191,000 cu m, a decrease in size of about 41 percent. Evaluation of the later surveys suggested that the disposal mound was migrating to the northwest along the normal bottom contours and that the surface irregularities observed in the earlier surveys were becoming less pronounced.

161. Data collected during storm periods in 1975 (maximum current velocities of 80 cm/sec) were used by Sternberg et al. (1977) to estimate the bed-load mass transport of sand. The storm episodes analyzed represented different levels of intensity, and as a result only approximations of the distances and quantities of sand movement were obtained. Table 6 lists the results of these computations.

162. Measurable quantities of suspended sediments were found in the study area during all seasons sampled. It has been well documented that the Columbia River is the most basic source of suspended sediment in the study area. The finer sediments in the study area move primarily as suspended load. Threshold conditions for these materials were exceeded 66 percent of the time

Table 6
Estimates of Mass Transport of Bed Load and Travel
Distance of Sand During Severe Storms of 1975
Columbia River, Oregon, Dredged Material Disposal Site

	Station 4	Station 5	Station 6	Station 6*
Dates	26-30 Aug	26-30 Aug	21-24 Dec	24-26 Dec
Mean wind speed, m/sec	13.5	13.5	14.8	10.0
Sediment modal size, φ units	2.75	2.75	2.50	2.50
Bed-load transport				
g/cm/storm	9.2	7.7	68.0	4.3×10^3
g/deposit/storm	4.8×10^5	2.5×10^5	3.5×10^6	2.2×10^8
cm ³ /deposit/storm	3.0×10^5	2.5×10^5	2.2×10^6	14.0×10^6
yd ³ /deposit/storm	0.4	0.3	2.9	183
Transport distance, m/storm	2.1	1.8	11.0	53

* These values represent about 55 percent of the total since data collection was interrupted midway through the passage of the storm.

during the winter storms. Bottom oscillatory currents produced by surface waves represent another means whereby bottom sediments are placed into suspension. Data collected during this study during various seasons show a high degree of variation of bottom sediment suspension by waves. It was determined that threshold conditions which cause bed formation (wave-generated ripples) required a minimum pressure fluctuation of 0.7 lb/in^2 (Boone et al. 1978).

163. It was obvious at the Columbia River, Oregon, dredged material disposal site, including the experimental disposal area, that wave activity will frequently initiate sediment movement. The coarser sediments characteristic of the experimental area, however, are not carried far from the seafloor by oscillatory wave-generated currents and tend to rapidly settle back to the bed. The net result of the combined effects of wave activity and other bottom currents will be that the dredged material deposit will gradually become smooth and spread toward the north-northwest. It is expected that this sedimentary feature will be relatively stable and bathymetrically recognizable for a few years.

Laboratory studies

164. Over the years, a tremendous amount of research in the form of laboratory tests has been conducted on the physical hydraulic aspects of sediment motion. Such tests have been conducted with sand, clay, and silt, under either unidirectional currents or scaled-wave climates. A few tests have been performed in the presence of both a unidirectional current and a superposed wave field, although these tests have certain inherent limitations unless the experiments are extremely well designed. Where laboratory tests to evaluate the physical characteristics of actual dredged material obtained from open-ocean disposal areas (sand, silt, and clay mixtures) with respect to their resuspension and transport capabilities are more limited than other tests of disposal site material, the results of such tests provide information pertinent to site capacity and retention capability. Thus, these tests have inherent direct application potential.

165. Galveston, Tex., disposed dredged material tests. Two-dimensional flume experiments were performed using four sand/silt/clay mixtures sampled from the offshore Galveston, Tex., dredged material disposal site in order to determine the critical erosion velocity, shear stress, and modes of sediment transport for each mixture. Also, an analysis of the hydrographic regime for offshore Galveston was performed based on meteorologic and oceanographic data collected between February 1975 and June 1976. Results of the flume experiments and hydrographic analysis were extrapolated to sediment transport processes believed operative in the offshore disposal site. Apparently, no studies other than one by Moherrek (1978) have been reported dealing directly with sediment mixtures containing near-equivalent amounts of sand, silt, and clay that have been obtained directly from a DMADS.

166. The input of each test sediment sample into the flume water column resulted in simultaneous turbidity plume generation and mounding of sediment on the flume bottom. The mounds exhibited a hummocky relief up to 20 cm high and were spread about 1.5 m upstream and downstream from the discharge point. All inputs occurred in still water except for one sample which was released into a 30-cm/sec flow. This resultant mound formed an elliptical configuration approximately 1.2 m downstream of the ejection point. Timing of the plume travel speed immediately after injection indicated an approximate head speed of 7.0 cm/sec.

167. Moherrek (1978) concluded: (a) each of the four sediments eroded similarly as evidenced by the similar critical bed shear values ($\tau_{cr} = 1.0 \text{ dyne/cm}^2$) necessary to initiate massive bed-load transport and rapid suspended concentration increases; (b) rapid deposition of suspended fine silt and clay occurred at very low bed shear values ($\tau_0 < 0.2 \text{ dyne/cm}^2$); (c) primarily silt and clay resuspension occurred during the initial 2 to 4 hr following each current velocity increase; thereafter, for sustained flow rates, the scour rate decreased to a near constant level; (d) continuous erosion on a hydrodynamically rough bed resulted in texturally diverse sediment interfaces within the sediment column; and (e) washload composition consisted primarily of fine silt and clay at a 30-cm elevation above the bed. Significant variations in the washload grain-size distribution occurred with increasing or decreasing flow rates.

168. Application of the experimentally determined critical erosion and deposition velocities as measures of the critical velocities occurring within the offshore disposal site is valid under certain assumptions. These assumptions include: (a) bottom currents in the offshore dredged material site are characterized by uniform, fully developed turbulent flow; (b) the effect of wave-induced oscillatory motion on the threshold of cohesive sediment movement is negligibly small; and (c) the hydrodynamic roughness exhibited by the dredged material placed into the flume is similar to the roughness existent on the offshore disposal mounds. Establishment of the validity and/or variability of these three assumptions requires the development of new field-monitoring techniques to measure the in-situ bottom current and sediment conditions over long time periods. Until such techniques are developed, extrapolation of the "critical velocity" results to field conditions should be viewed as a best estimate based upon state-of-the-art methods.

169. It should be noted that the experimental flume tests with the samples of dredged material from the open-ocean disposal site at Galveston, Tex., were conducted with unidirectional flow only, and without benefit of wave or storm effects. Although no hurricanes seriously affected the Galveston vicinity during the period of study by Moherrek (1978), destructive storms such as Hurricane Audrey (1957), Carla (1961), and more recent storms have produced extreme storm tides. Hurricane Carla caused a 2.3-m tidal height rise. After passage of the storm inland, seaward-flowing density currents deposited a fresh sand layer (up to 3 cm thick) over a sandy mud bottom in depths of

15 to 20 m. In addition, a graded bed (up to 10 cm thick) composed of very fine sand, silt, and clay was traced over the homogeneous mud bottom to depths exceeding 60 m. Such sedimentologic features point out the major role that a tropical storm can have in modifying an offshore geologic environment over a very short period of time.

170. Taunton River and Thames River dredged material test. Threshold erosional velocities and rates of erosion were determined by Gularte et al. (1977) for material dredged from the Taunton River, Massachusetts, and Thames River, Connecticut, prior to being placed in an open-ocean disposal site. Both of the dredged materials tested can be classified as marbled black-olive drab, fairly well-graded silty clay of high organic content. The Taunton River sediments contained approximately 20 percent (by weight) clay-size particles (less than 0.002 mm) and 70 percent silt-size particles (0.06 to 0.002 mm), with the remainder fine sand and shell fragments. The material dredged from the Thames River contained much less clay-size particles (10 percent), slightly less silt-size particles (60 percent), and 30 percent sand-size material and shell fragments. The dredged material was tested in a research water tunnel designed and fabricated specifically for this research.

171. Results of the tests conducted on the Taunton and Thames River dredged material are shown in Figure 22. In these tests the velocities were increased linearly at a rate of 24.4 and 21.3 cm/sec/hr for the Taunton and Thames River materials, respectively, up to a maximum velocity of approximately 75 cm/sec. The dredged material was subjected to small step increases in water velocity; turbidity was allowed to stabilize and, after a constant turbidity was achieved, the average velocity was increased. This procedure was continued until either a maximum turbidity or the maximum tunnel velocity was reached. These tests were used to determine the minimum linear rate of velocity increases. Subsequent experiments were made at rates less than these minimums to ensure that erosional equilibrium was reached before the velocity was increased.

172. From Figure 22 it can be seen that, in both cases, the amount of material suspended increased essentially linearly, with increasing velocity up to about 30 cm/sec for the Taunton River and up to about 50 cm/sec for the Thames River, above which the amount of material suspended increased markedly. The threshold erosional velocity was taken as the velocity above which there was a marked increase in the amount of material suspended. The threshold

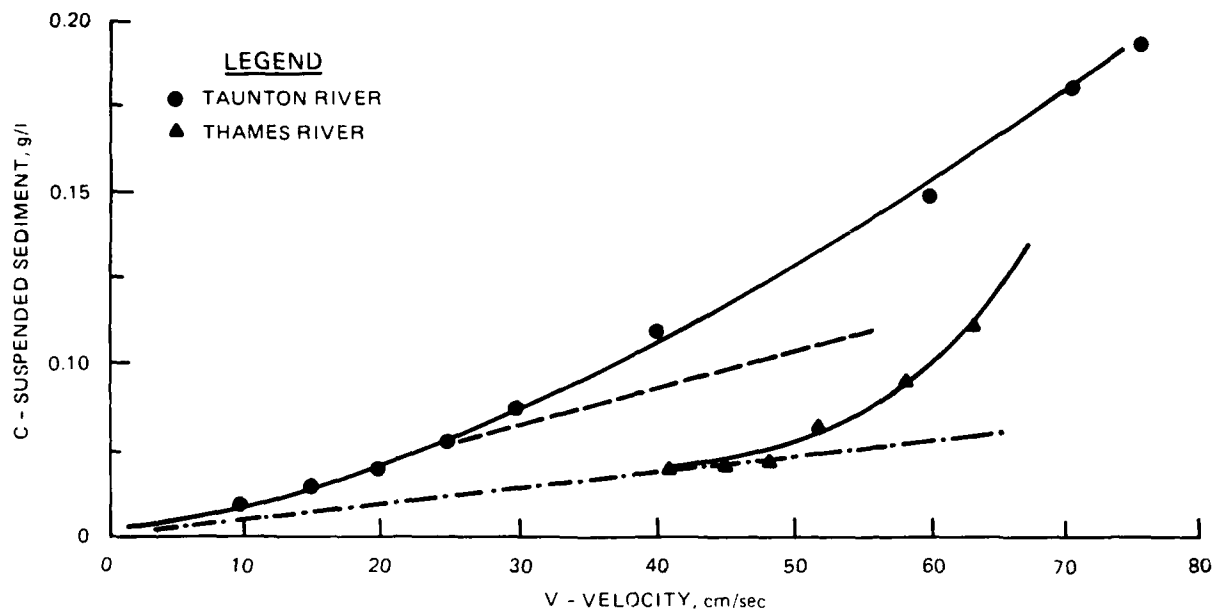


Figure 22. Material suspended as a function of average velocity, Taunton River Massachusetts, and Thames River, Connecticut (after Gularte et al. 1977)

shear stress (corresponding to the threshold velocities) for the Taunton River was found to be 2.14 dynes/cm² and 3.14 dynes/cm² for the Thames River dredged material.

173. The empirical expressions for concentrations of suspended material as a function of velocity were, for the Taunton River:

$$C = 2.55 \times 10^{-4} V^{1.46} \quad (42)$$

For the Thames River:

$$C = 1.00 \times 10^{-8} V^{3.78} \quad (43)$$

where

C = concentration of material suspended, g dry material eroded/l of seawater

V = average velocity of the water above the exposed sediment, cm/sec

174. When the above expressions are differentiated with respect to velocity and multiplied by the rate at which the velocity was increased, the expression for the time rate at which material is suspended as a function of

velocity is obtained. For the Taunton River material, the rate of erosion, de/dt , as a function of average shear stress is:

$$\frac{de}{dt} = 1.17 \times 10^{-6} \tau^{0.269} \quad (44)$$

For the Thames River:

$$\frac{de}{dt} = 1.93 \times 10^{-7} \tau^{1.63} \quad (45)$$

where

e = amount of material eroded, g dry weight/sq cm of eroded surface

τ = average shear stress, dynes/sq cm

These results are presented in Figure 23. Here it can be seen that the amount of material eroded is a continuous function of average horizontal shear stress (Gularte et al. 1977).

175. Resuspension of deposited kaolinite sediment beds. Mehta and Partheniades (1982) performed a series of laboratory investigations to define the erosional behavior of deposited cohesive sediment beds in flumes using kaolinite. Under a given bed shear stress, erosion occurs at a continuously decreasing rate up to a depth at which the bed shear stress equals the shear strength. The bed shear stress is therefore also equal to the critical shear stress for erosion at that depth. An expression relating the rate of erosion to the difference between the bed shear stress and the critical shear stress has been obtained. The critical shear stress was found to increase both with depth and with the bed consolidation time. The rate of erosion decreases with increasing consolidation time.

176. From the laboratory investigation of deposited cohesive sediment beds in flumes using kaolinite, Mehta and Partheniades (1982) determined that:

- a. The rate of surface erosion of stratified, deposited beds continuously decreases with time and can even become zero as the depth of erosion increases. On the other hand, the erosion rate of uniform beds remains practically invariant with depth of erosion.
- b. The decrease in the erosion rate of stratified beds occurs because the cohesive shear strength with respect to erosion of the bed increases with depth. During erosion, flocs are detached from the bed and entrained, but redeposition of the entrained sediment does

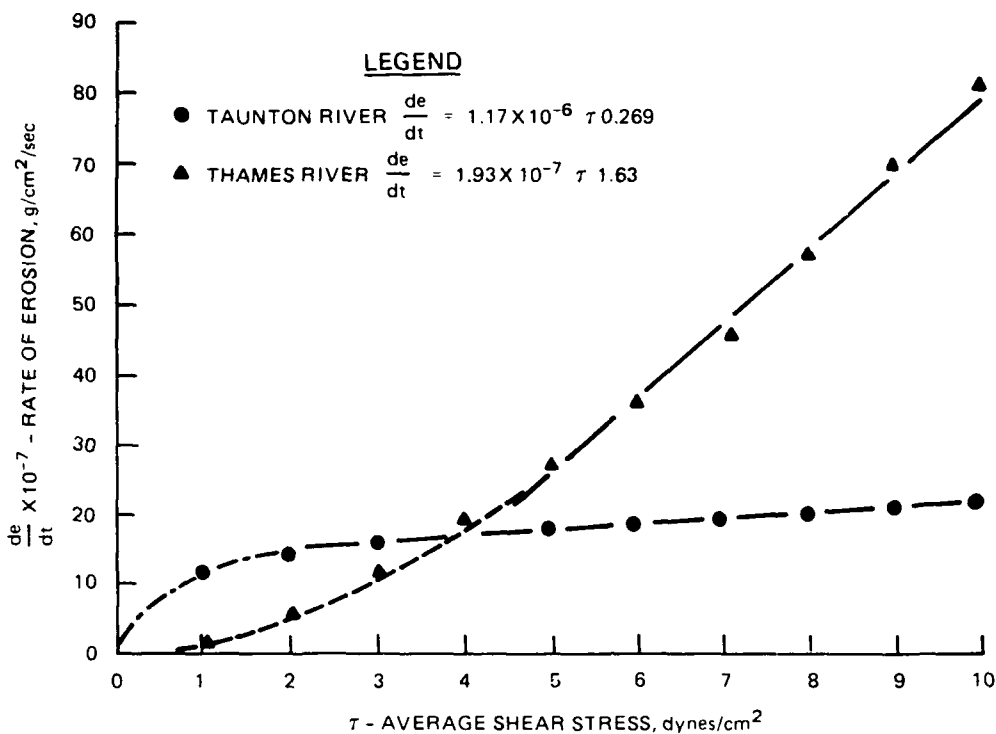


Figure 23. Rate of erosion as a function of average shear stress, Taunton River, Massachusetts, and Thames River, Connecticut (after Gularte et al. 1977)

not occur. Erosion is arrested at a depth where the bed shear stress, τ_h , equals the bed shear strength. This value of τ_h is equal to the critical shear stress, τ_c , of the bed at that depth.

c. An expression for the rate of surface erosion was determined:

$$\frac{e}{e_o} = \exp \alpha \frac{\tau_b - \tau_c}{\tau_c} \quad (46)$$

where e_o and α are empirical coefficients. The value of α ranged from 5.6 to 15.0, and e_o from 0.000038 to 0.000153 g/cm²/min. The rate varies exponentially with the normalized excess shear stress, $(\tau_b - \tau_c)/\tau_c$.

d. The critical shear stress in general increases with depth below the initial bed surface and also increases with bed consolidation time. As a result, the rate of erosion decreases with increasing consolidation time.

PART IV: METHODS OF ANALYSIS FOR TRANSPORT AND REDEPOSITION OF MOUND MATERIAL

177. The transport of resuspended dredged material from a disposal mound, and the ultimate redeposition of that material from the water column, is governed by an extension of the same physical forces which initially caused the resuspension from the mound. Redeposition of mound material occurs when the supply of depositable sediment exceeds the flow's transport; thus, a region in sedimentation equilibrium is one in which the flow is just able to prevent quasi-permanent deposition of the available supply of sediment. A change in deposition behavior at a site can be the result of an increase/decrease in depositable sediment and/or an increase/decrease in transport capability.

Steady-State Analytical Methods

178. Hydrodynamic conditions at the site are used to calculate directions (usually at least two) and rates of transport out of the site. These hydrodynamic conditions along the projected path must be postulated so that zones of deposition can be identified. In tidal flows, sediment may easily deposit temporarily and then be resuspended again during strength of flow. Because of the oscillatory nature of tidal currents, the resuspended sediments may move in and out of the disposal site with the strength of the tidal flow. Describing this process analytically requires a more detailed knowledge of the hydrodynamic conditions than is usually available. Therefore, a steady-state analytical approach to transport and redeposition is reduced to the following steps.

- a. Determining the path that the resuspended sediment will follow.
- b. Calculating the distribution (depositional capability) of sediment (suspended and bed load) along the path.
- c. Concluding that the redeposited sediment thickness is either negligible or significant.

179. Apparently no reports exist in the technical literature of analytical investigations that were conducted explicitly for the purpose of ascertaining the ultimate disposition of material that had been resuspended and transported from a dredged material aquatic disposal site. However, an analytical approach could be developed, such as the one presented here.

The transport path

180. The dominant direction of flow in an estuary or bay can be approximated by consideration of the ebb and flood channels and by the freshwater inflow locations. When it becomes necessary to establish a disposal site in a bay or estuary (i.e., San Francisco Bay), an evaluation of the effect of boundary formation eddies should be performed. In such an area, the deviation of flow lines from the dominant direction will induce vortex shedding, which will eventually spin down to a velocity less than that required for suspension and transport, and any material in suspension and transported by the vortex can then be deposited.

181. For steady-state evaluation, all parameters are considered constant with respect to time and deposition. That is to say, the disposal mound is considered to be placed in the flow field instantaneously, and potential erosion exists until the mound has disappeared or until the mound has armored to such an extent that erosion is no longer possible from the steady-state hydrodynamic conditions that prevail.

182. The flow velocities at locations in an estuary or bay can be approximated from hydrographic surveys that provide cross-sectional flow area and discharges into such a closed system. When the aquatic disposal site is positioned in the open ocean or at a location in a bay or estuary so that it will be subjected to the oscillation velocities of alternating tidal flow, the dominant flow velocity that will be available for transport should be considered. When transport occurs from a flood flow into an estuary, any material that may be deposited in the estuary could be in a location such that the alternating ebb current will not have sufficient magnitude for resuspension of this same material. Shoaling will result in this specific location.

Assumed sediment distribution

183. Steady-state evaluations arise by a comparison of the available stresses with the required stresses for transport at successive cross sections along the flow paths. As the flow intensity decreases, the larger particles will settle from the water column. Successive decreases in flow velocity and turbulence parameters will ultimately permit the settling of all but the finest particle sizes. A priori knowledge of the volume of material available for transport and the sediment size distribution of the resuspended material will permit an estimation of the thickness of the accumulated material at successive cross sections. It can be logically assumed that a uniform

distribution of material thickness will exist between these computation sections if discontinuities or otherwise abrupt variations in the flow field do not exist.

Conclusions regarding
deposited sediment thickness

184. Both steady-state ebb and steady-state flood currents should be evaluated in this analysis, with the understanding that any resulting deposition will be the net effect of a long-term average condition. In the simplest case, the deposited sediment distribution should asymptotically decline in a spatial direction away from the source until other currents are encountered that will reduce the rate of deposition or preclude such deposition entirely. The conceptual process described has been schematized by Morton and Robertson (1983) as shown in Figure 24.

Example steady-
state analytical procedure

185. Long-term transport contours have been calculated and mapped by George and Walton (1984) to illustrate the far-field dilution associated with unit concentrations of a dredged material source under conditions of ambient advection and dispersion. The procedure is to use closed-form, two-dimensional horizontal solutions of the steady-state mass transport equation for both point and distributed sources. If the mean net surface drift is less than the mean magnitude of the tidal current parallel to it (x-direction) over half of the tidal cycle, then overlapping of various plumes might be expected and a continuous (steady-state) release plume model should be used. Otherwise, a discrete (unsteady) source plume model should be used. To estimate dispersion coefficients in the x- and y-directions, the tidal excursion in these directions is needed.

$$L_x = U_{tx} * \frac{T}{2} * \frac{\pi}{4} \quad (47)$$

and

$$L_y = U_{ty} * \frac{T}{2} * \frac{\pi}{4} \quad (48)$$

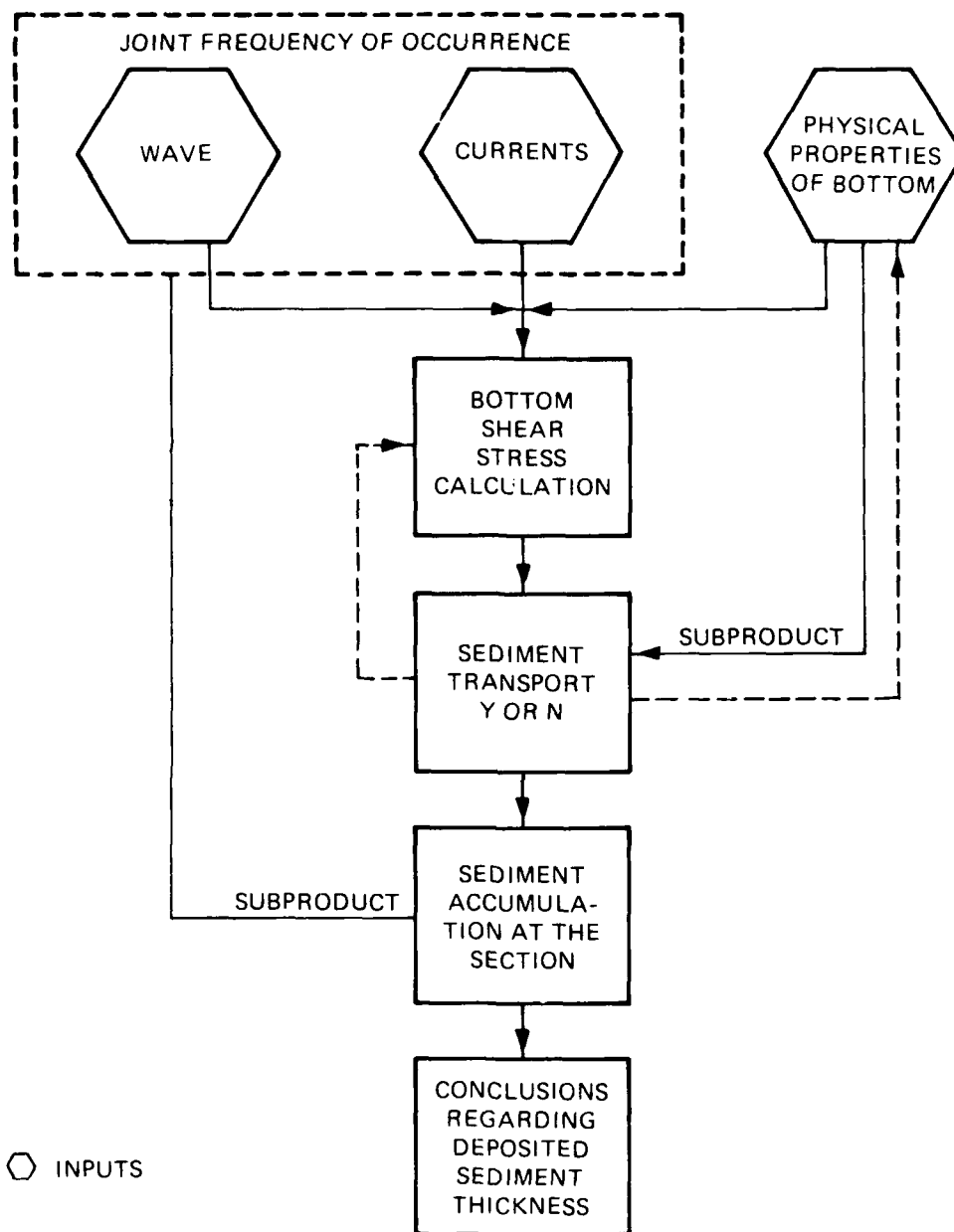


Figure 24. Schematic analysis of transport and redeposition of mound material (after Morton and Robertson 1983)

where

- L_x = tidal excursion parallel to mean net surface drift, m
- U_{tx} = maximum tidal velocity parallel to mean net surface drift, m/sec
- T = tidal period, sec
- L_y = tidal excursion perpendicular to mean net surface drift, m
- U_{ty} = maximum tidal velocity perpendicular to mean net surface drift, m/sec

Using the tidal excursion lengths, L_x and L_y , dispersion coefficients can be estimated in the x- and y-directions using a conservative simplification of the so-called "four-thirds" rule.

$$D_x = 0.0018 * L_x \quad (49)$$

and

$$D_y = 0.0018 * L_y \quad (50)$$

where D_x and D_y are dispersion coefficients parallel and perpendicular to the mean net surface drift direction, respectively, in square metres per second.

186. If the mean net surface drift, U_m , is less than the mean tidal velocity ($U_{tx} * \pi/4$), the site is assumed to behave as a continuous point-source under successive disposals. In this case, a two-dimensional, continuous, point-source plume model with unit loading ($q = 1$) should be used to develop characteristic concentration contours:

$$C(x,y) = \left[\frac{q}{d * (4\pi * D_y * U_m * x)^{1/2}} \right] * \exp \left[-\left(y^2\right) * \frac{U_m}{4 * D_y * x} \right] \quad (51)$$

where

- $C(x,y)$ = concentration at coordinate (x,y), with (0,0) as point of disposal
- q = mass input rate/sec (= 1 for unit contours)

d = calculation depth, m ($d = h_p$ if above the pycnocline,
 $d = h - h_p$ if below the pycnocline; or $d = h$ if entire
 water depth is used)

h_p = pycnocline depth, m

h = mean water depth, m

D_y = dispersion coefficient perpendicular to mean net surface drift,
 sq m/sec

U_m = mean net surface drift, m/sec

An example of concentration contours for a unit loading is presented in Figure 25. For this particular example, the contours were developed by calculating and plotting the concentrations for several sets of (x,y) coordinates on a rectangular grid. Then the concentration contours were formed by interpolating between the calculated values for lines of equal concentration.

187. The disposed material will accumulate on the bottom if the threshold velocity for material movement, V_t , is greater than the total maximum bottom fluid velocity, U_{tot} . The total maximum bottom velocity is estimated as the summation of the mean net bottom drift, U_m , the maximum tidal velocity, U_{tx} or U_{ty} , and the maximum orbital-wave bottom velocity.

$$U_{tot} = U_{bm} + U_t + U_w \quad (52)$$

where

U_{tot} = total maximum bottom velocity, ft/sec

U_{bm} = mean net bottom drift, ft/sec

U_t = maximum of U_{tx} or U_{ty}

U_w = maximum orbital-wave bottom velocity, ft/sec

Hence, if $V_t > U_{tot}$, the site will tend to retain deposits. If $V_t < U_{tot}$, the site will tend to be dispersive.

188. During those portions of the tidal cycle when slack-water conditions exist (e.g., when the tidal current is reversing from flood to ebb, or vice versa), the total maximum bottom fluid velocity may become too small to continue moving material that is in transport. In this case, material that was in transport will be deposited on the bottom. As the tidal velocity increases in the opposite direction, the material may again be placed in suspension and carried in a different direction. The ultimate location of the material will be determined by the divergence of the flow field with respect

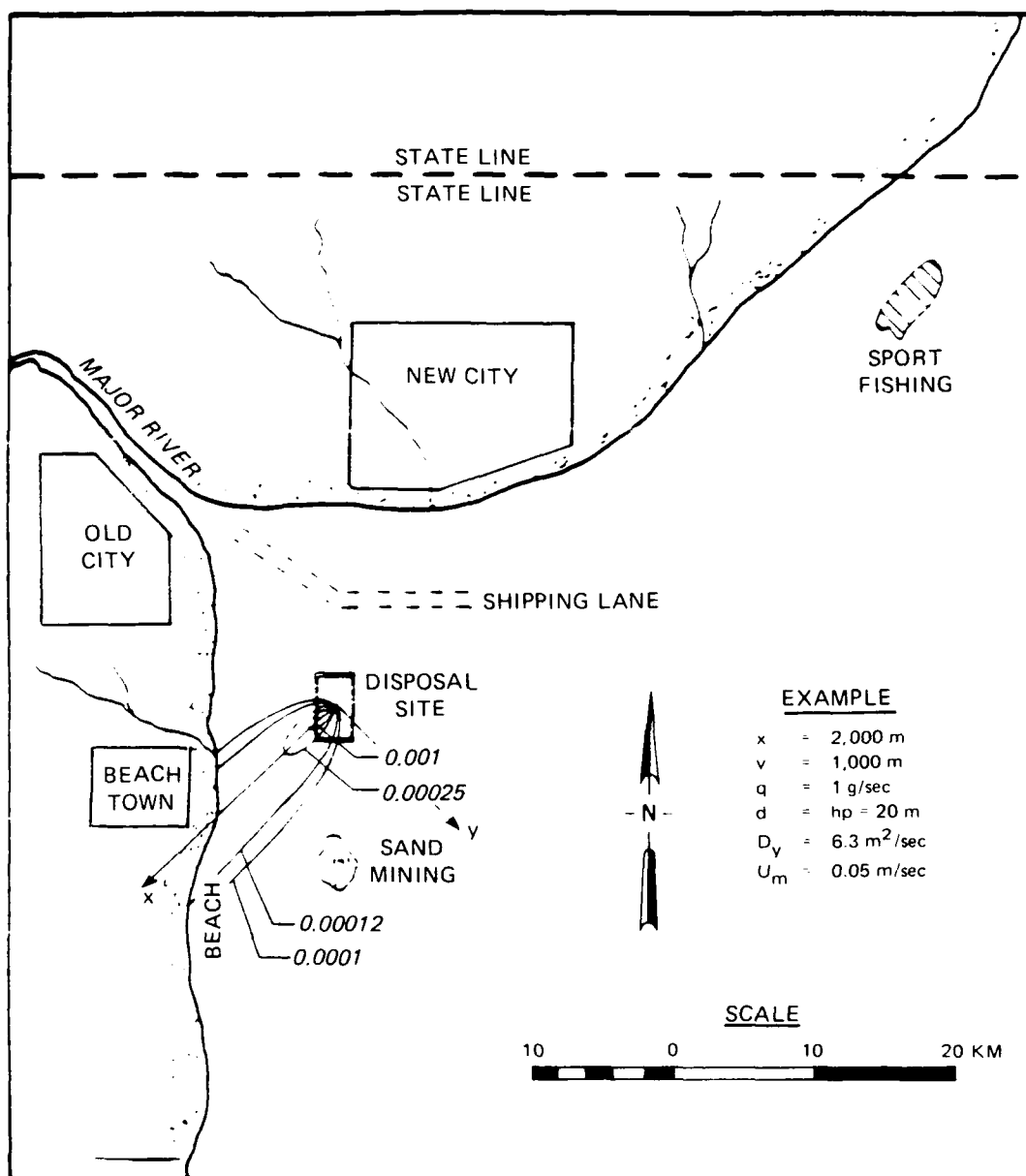


Figure 25. Contours of concentration for unit load at point of disposal, assuming continuous point source (after George and Walton 1984)

to the net transport capacity for each portion of the tidal cycle. The potential for resuspension exists when the total maximum bottom fluid velocity exceeds the threshold velocity for material movement.

189. The two-dimensional, continuous point-source plume model provides the dilution contours under steady-state current conditions (average values). When the resuspended material is transported to a region of the flow field where conditions of current velocity become less than the average values, the material being transported in the suspension plume will settle from the water column to the bed, where it will remain until stresses again become sufficient for resuspension and transport to another location.

Time- and Rate-Dependent Analytical Methods

Discretization and sequencing of events

190. It is not apparent that any specific time- and rate-dependent analytical investigations have been conducted regarding the transport and redeposition of resuspended disposal mound material expressly for the purpose of ascertaining its ultimate fate. However, time and rate analyses are simply an extension of the steady-state considerations under the added stipulations that velocities may vary with time at the particular locations of interest.

191. Also, inherent in the analysis is the added assumption that additional material may be placed in the disposal area with time. Thus, the time- and rate-dependent analysis becomes one of incremental calculations of deposition potential and accumulation at specific cross sections in the flow field at finite increments of time during which the current velocities may have varied and the sediment concentrations in the water column may have been altered. Under these considerations, the material previously deposited at a section under low current velocities may be again resuspended and transported from such a section to another section in the flow field. Hence, an accounting must be performed of the conditions at each section. Such analytical methods can be easily computerized for rapid data processing.

192. The discretization and sequencing of events fundamental to an analysis of time- and rate-dependent processes responsible for mound resuspension are also necessary for analyzing transport from the disposal site and redeposition in another location. The processes of tide elevation and tidal current velocities are continuous functions that vary with time, position, and

rate. It is necessary to digitize these continuous processes into a series of distinct elements, to select those discrete values of the various energy sources at the appropriate time, and to simulate the processes at a definite series of time events or unit steps of the rate changes which may be representative. Morton and Robertson (1983) have schematized this conceptual analysis for a specific storm event at a particular cross section of an estuary or bay or across any arbitrary current flow pattern in the open ocean. This schematization may be applied equally as well over a series of tidal cycles or other temporal phenomena (Figure 26).

Example time- and rate
dependent analytical procedure

193. The work of George and Walton (1984) was extended to investigate time- and rate-dependent analyses of far-field dilution associated with two-dimensional, discrete point sources and two-dimensional, discrete distributed source models. The procedure again is to provide closed-form, two-dimensional, horizontal solutions of the transport equation for both point and distributed sources. Here again, redeposition of the suspended materials being transported by the plume will occur when the stresses determined by the total bottom velocity components become less than the threshold values for material transport. This situation will occur in regions of flow divergence and at times of slack water during the tidal cycle. The time- and rate-dependent analysis becomes an incremental calculation of the deposition potential and accumulation at specific locations in the flow field at specified times of the tidal cycle. Here again, settling velocity and time are inherently accounted for in the assumptions pertaining to turbulence and mixing stresses necessary for resuspension, transport, and redeposition.

194. Two-dimensional discrete point source model. If the mean net surface drift, U_m , is greater than the mean tidal velocity, $(U_{tx} * \pi/4)$, the site is assumed to behave as a series of discrete loadings that never merge. In this model, the source is also assumed to be small enough to be considered a point source when viewed from the far field. The following two-dimensional equation describes the horizontal movement and dispersion of the plume through time:

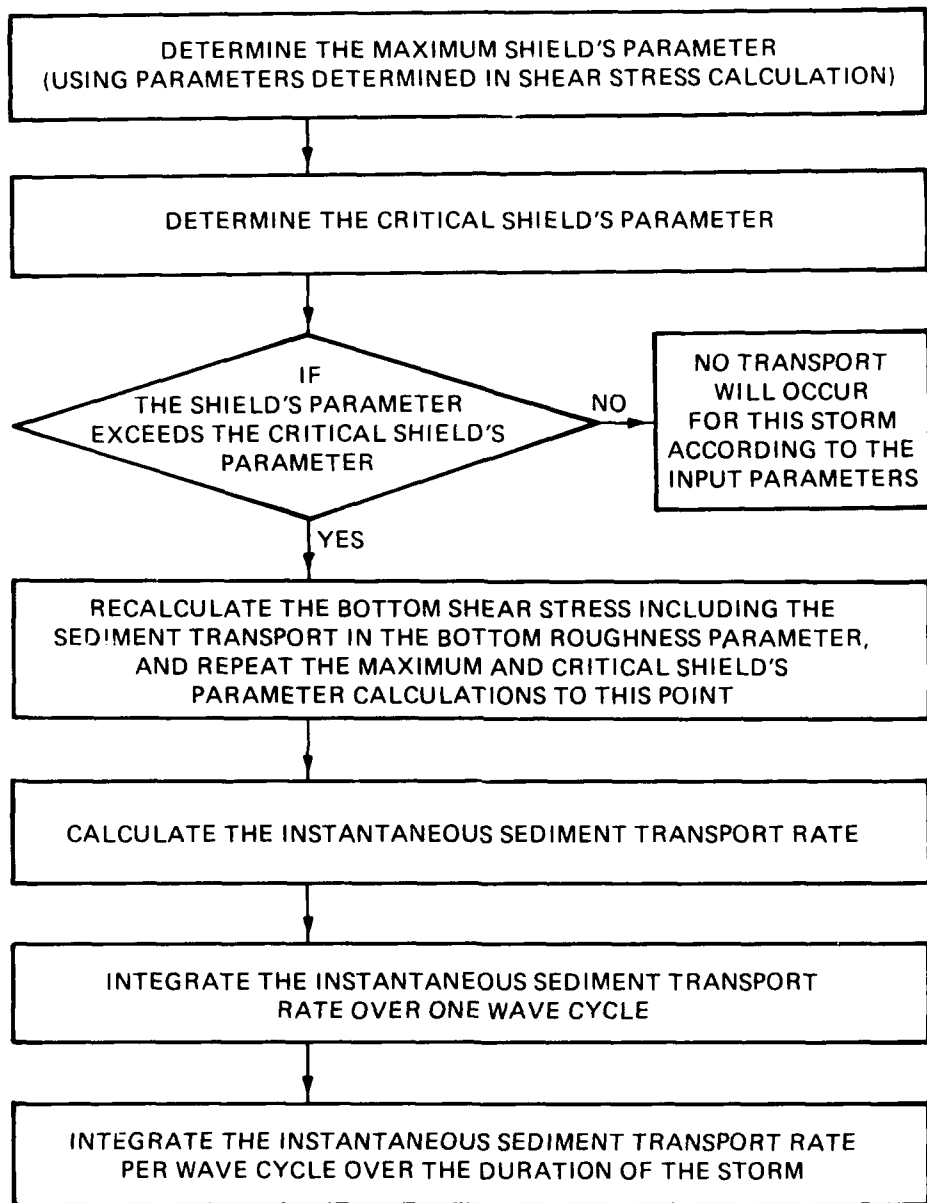


Figure 26. Schematic analysis of transport and redeposition of mound material for a storm situation (after Morton and Robertson 1983)

$$C(x,y,t) = \frac{Q}{4\pi * d * (D_x * D_y)^{1/2} * t} * \exp \left[\frac{-(x - U_m * t)^2}{4 * D_x * t} - \frac{y^2}{4 * D_y * t} \right] \quad (53)$$

where

$C(x,y,t)$ = concentration at coordinates (x,y) at time t , with $(0,0,0)$ as the point and time of disposal

Q = mass unit (= 1 for unit contours)

D_x, D_y = dispersion coefficients parallel and perpendicular to the mean net surface drift direction, respectively, sq m/sec

195. An example of concentration contours for a unit load at times of 1, 5, and 10 hr is given in Figure 27, with the positive x-direction parallel to the mean net surface drift, U_m , and the y-direction perpendicular to the x-axis. For this particular example, the contours were developed by calculating and plotting on a rectangular grid the concentrations for several sets of (x,y) coordinates at the three different times. Then, the concentration contours were formed by interpolating between the calculated values for lines of equal concentration. For this example, the concentrations, the (x,y) coordinates, and the various times were chosen to produce the three separate sets of contours as shown in Figure 27.

196. Two-dimensional discrete distributed sources model. If the mean net surface drift, U_m , is greater than the mean tidal velocity and if the source is assumed to have finite dimensions (distributed by a moving dump), the following two-dimensional equation describes the horizontal movement and dispersion through time (George and Walton 1984).

$$C(x,y,t) = \frac{C_o}{4} * \left\{ \operatorname{erf} \left[\frac{\frac{b}{2} - x + U_m * t}{(4 * D_x * t)^{1/2}} \right] + \operatorname{erf} \left[\frac{\frac{b}{2} + x - U_m * t}{(4 * D_y * t)^{1/2}} \right] \right\} * \left\{ \operatorname{erf} \left[\frac{\frac{a}{2} - y}{(4 * D_y * t)^{1/2}} \right] + \operatorname{erf} \left[\frac{\frac{a}{2} + y}{(4 * D_y * t)^{1/2}} \right] \right\} \quad (54)$$

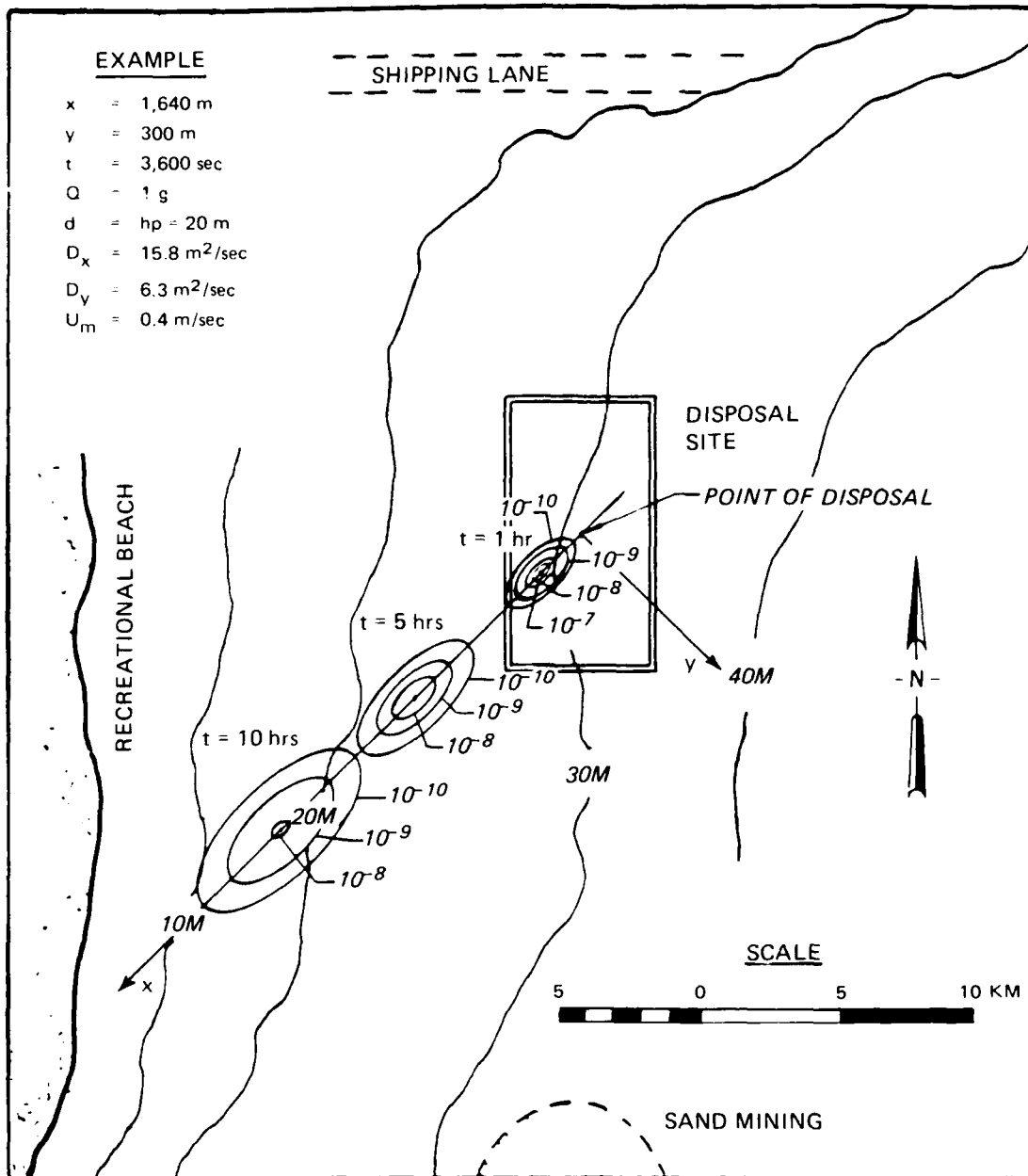


Figure 27. Contours of concentration for unit load at point of disposal, assuming discrete point source (after George and Walton 1984)

where

C_o = initial concentration (= 1 for unit contours)

erf = error function (Table 7)

$b = v * t'$ (initial plume dimension along the x-axis), m

v = disposal vessel speed, m/sec

t' = individual disposal duration, sec

$a = 2 * w$ (initial plume dimension along the y-axis), m

w = disposal vessel width, m

197. An example of concentration contours, for a unit concentration, at times of 0.25 and 0.75 hr is given in Figure 28, with the positive x-direction parallel to the mean net surface drift, U_m , and the y-direction perpendicular to the x-axis. For this particular example, the contours were developed by calculating and plotting on a rectangular grid the concentrations for several sets of (x,y) coordinates at the two different times. Then, the concentration contours were formed by interpolating between the calculated values for lines of equal concentration. For this example, the concentrations, the (x,y) coordinates, and the various times were chosen to produce the two separate sets of contours as shown in Figure 28 (George and Walton 1984).

Modeling

Physical modeling of transport and redeposition

198. Fixed-bed hydraulic models have long been recognized as an extremely valuable tool in studying the effects of coastal or estuarine construction projects on wave, tide, and current conditions. In recent years, the use of relatively small quantities of sediment tracer material in fixed-bed models has generally been accepted as the most reliable and least expensive method of studying sediment transport and deposition due to wave and tidal action. In practically all cases, the results of such studies are considered qualitative rather than quantitative.

199. In general, movable-bed model similitude laws require distorted scales. However, these laws can be adapted to undistorted-scale, fixed-bed models for the selection of tracer materials; the scaling relations of Noda (1972) are used as an example (Figure 29).

Table 7
Error Function (erf) and
Complementary Error Function (erfc)

$$\text{erf}(\beta) = \frac{2}{\sqrt{\pi}} \int_0^{\beta} e^{-\epsilon^2} d\epsilon$$

$$\text{erf}(-\beta) = -\text{erf}\beta$$

$$\text{erfc}(\beta) = 1 - \text{erf}(\beta)$$

<u>β</u>	<u>erf (β)</u>	<u>erfc (β)</u>
0	0	1.0
0.05	0.056372	0.943628
0.1	0.112463	0.887537
0.15	0.167996	0.832004
0.2	0.222703	0.777297
0.25	0.276326	0.723674
0.3	0.328627	0.671373
0.35	0.379382	0.620618
0.4	0.428392	0.571608
0.45	0.475482	0.524518
0.5	0.520500	0.479500
0.55	0.563323	0.436677
0.6	0.603856	0.396144
0.65	0.642029	0.357971
0.7	0.677801	0.322199
0.75	0.711156	0.288844
0.8	0.742101	0.257899
0.85	0.770668	0.229332
0.9	0.796908	0.203092
0.95	0.820891	0.179109
1.0	0.842701	0.157299
1.1	0.880205	0.119795
1.2	0.910314	0.089686
1.3	0.934008	0.065992
1.4	0.952285	0.047715
1.5	0.966105	0.033895
1.6	0.976348	0.023652
1.7	0.983790	0.016210
1.8	0.989091	0.010909
1.9	0.992790	0.007210
2.0	0.995322	0.004678
2.1	0.997021	0.002979
2.2	0.998137	0.001863
2.3	0.998857	0.001143
2.4	0.999311	0.000689
2.5	0.999593	0.000407
2.6	0.999764	0.000236
2.7	0.999866	0.000134
2.8	0.999925	0.000075
2.9	0.999959	0.000041
3.0	0.999978	0.000022

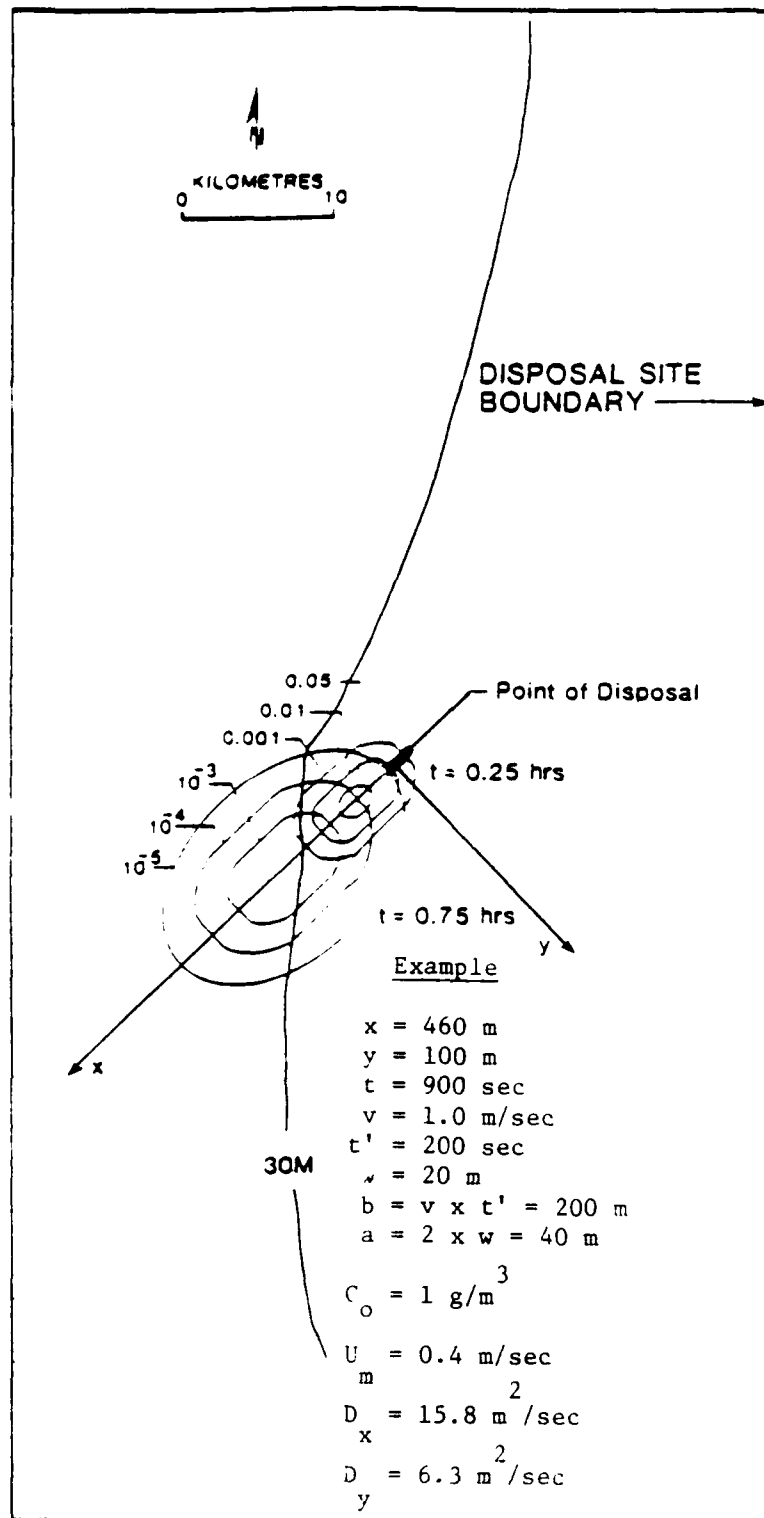


Figure 28. Contours of concentration for unit load at point of disposal, assuming discrete distributed sources (after George and Walton 1984)

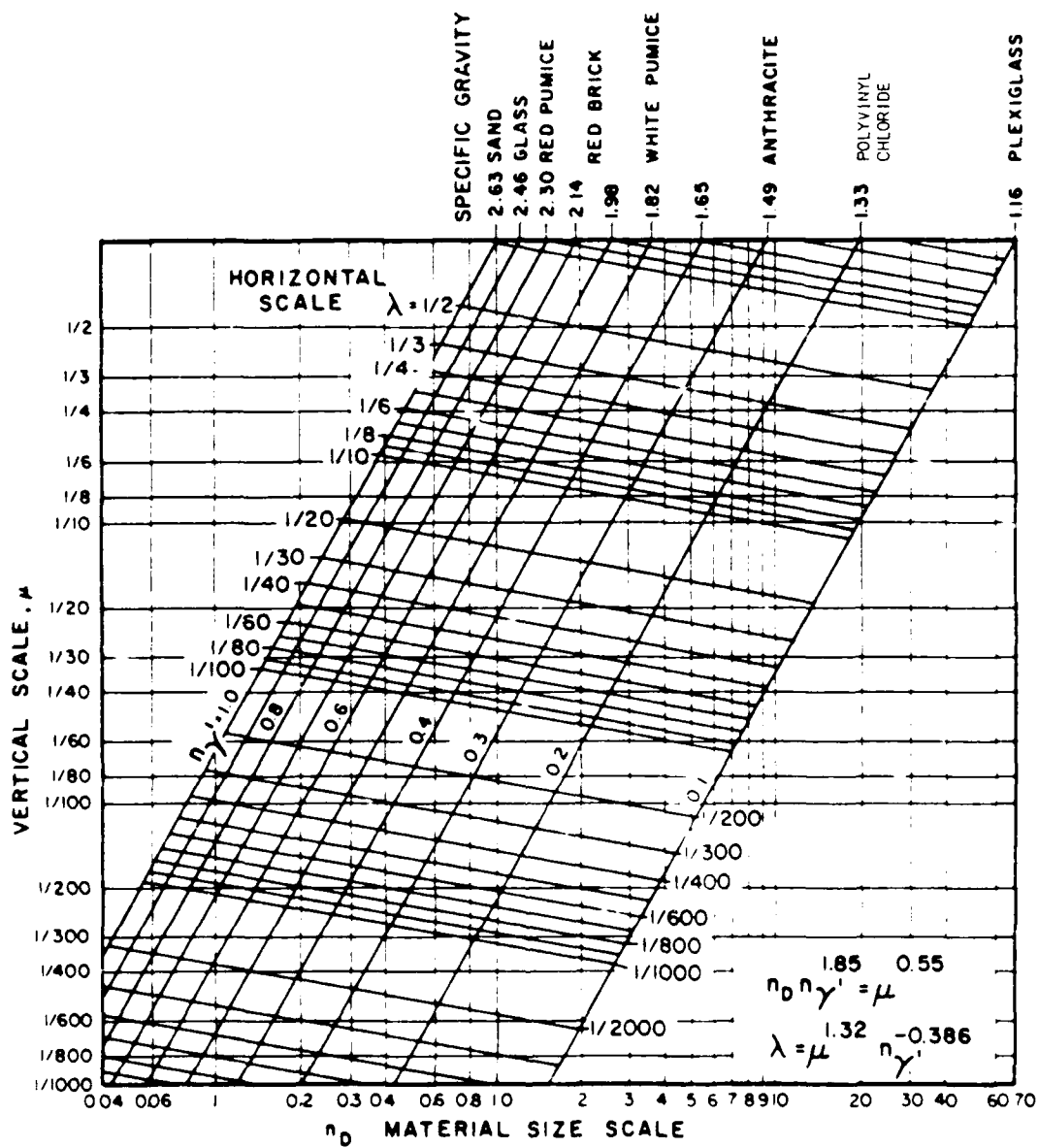


Figure 29. Graphic representation of model law (after Noda 1972)

200. Noda (1972) indicates a relationship or model law among the four basic scale ratios: horizontal scale, vertical scale, sediment size, and relative specific-weight ratio. These relationships were determined experimentally from a wide range of wave conditions, current velocities, and materials. This appears to be an appropriate scale relation for studying the movement of dredged material in an open-ocean environment and has proven effective in evaluating the movement and subsequent deposition of sediment in several three-dimensional model investigations at the US Army Engineer Waterways Experiment Station (Whalin and Chatham 1979) in which wave action was the primary energy source for sediment transport.

201. The basic objective of fixed-bed model shoaling verification is to identify a synthetic sediment that will move and deposit under the influence of the model forces in the same manner that the natural sediment moves and deposits under the influence of the natural forces. Because no satisfactory similitude laws have been developed for estuarine or oceanic sedimentation, the development of the model shoaling procedure is more an art than a science at this time. The appropriate time and volume scales for the shoaling tests must be determined by trial and error.

202. Many variables are involved in identifying a suitable operating technique for use in the model, and each must be resolved by trial and error in the model. The most significant variables include: (a) shape, size, gradation, and specific gravity of the synthetic sediment; (b) method, location, duration, and quantity of synthetic sediment injection; (c) rate of freshwater inflow; (d) magnitude of tide; (e) height, direction, and period of ocean waves; (f) length of model operation; and (g) readjustment of model roughness. The model water temperature must be closely monitored since similar shoaling tests performed with different water temperatures can give significantly different results.

203. Because the shoaling test technique is developed by trial and error, the validity of the shoaling verification is highly dependent on the quality and quantity of the available prototype data. Surveys of the problem area should be available for a period of at least 2 and preferably 3 or more years, in order that average annual conditions can be determined. The problem area is subdivided into sections, and the average annual prototype shoaling rate is determined for each section. These rates are then converted to percentages of the shoaling rate for the entire problem area, and this is the

percentage distribution that is reproduced in the model. When an acceptable reproduction of the distribution pattern has been achieved, the volume of material recovered from the problem area in the model can be equated to the prototype shoaling rate to establish an approximate shoaling volume scale. The duration of the model test can also be equated to the prototype period for which the shoaling rate was developed to determine the shoaling test time scale.

Numerical modeling of transport and redeposition

204. The hydrodynamic and sediment transport models previously considered for the resuspension of dredged material placed at an aquatic disposal site can also be used to determine the location at which this resuspended material will eventually be redeposited. Sediments enter into the coastal environment via freshwater inflow during high runoff periods, erosion of the shoreline, and disposal of dredged material in a resuspension from the DMADS. Generally, relatively fine clay and silt particles comprise a major portion of the sediment, along with sandy constituents. Such sediment can be transported by coastal currents over some distance before being redeposited on the ocean or estuary bottom. Strong coastal currents and high waves can cause significant bottom shear stress and, hence, entrain bottom sediment. Usually, complicated entrainment-transport-deposition cycles lead to large spatial and temporal variations in sediment distribution within any specific coastal region and environment. A comprehensive impact analysis is required to understand the far-field fate of dredged material after it is transported from the DMADS. The complex interactions occurring among various components of a coastal ecosystem have been presented by Sheng (1983) (Figure 30).

205. Existing comprehensive modeling studies of coastal currents and sediment transport on a large spatial scale, incorporating all the major components shown in Figure 29, are relatively scarce. Computational cost increases in direct proportion to the number of computational cells requiring simulation; hence, most investigations involving the removal of dredged material from the disposal site cease computation when the material has passed the computational grid boundary and has left the immediate region of interest. This is partly due to the lack of detailed understanding of the complex hydrodynamic and sedimentary processes and partly due to the lack of efficient mathematical models suitable for long-term simulations.

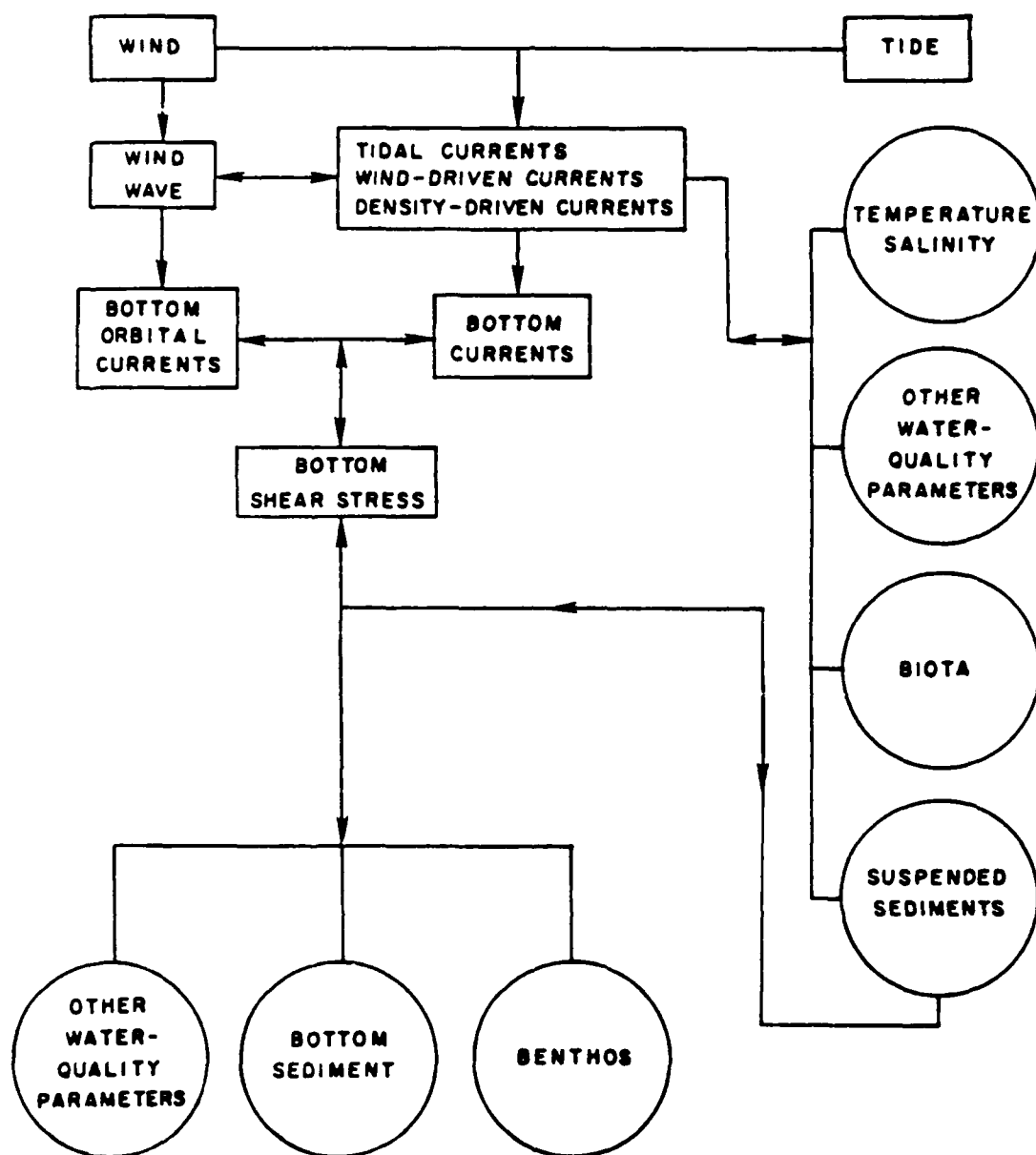


Figure 30. Schematic of dynamic interactions among various components of a coastal ecosystem in the vicinity of a DMADS (after Sheng 1983)

206. After entering the water column as influx or resuspension from the bottom, the movement of sediment is influenced by the various transport modes including convection, turbulent mixing, and gravitational settling. Exchange between the suspended sediment and the bottom sediment is described by the entrainment (resuspension) and deposition models. The transport of sediment in the water column depends on the properties of the sediment, the physico-chemical properties of the fluid, and the turbulence and mean currents of the flow field. In general, a particle-size distribution exists at any specific point in the water column and, due to the dynamic nature of the flow field, this distribution is usually a function of time.

207. If the time scales of the dominant forcing functions (e.g., wind waves, tidal current) are larger than the other characteristic time scales of the water body, a quasi-steady state exists and one may use a steady-state model that removes the effect of time variation from the problem. It is also convenient to use the steady-state models to study a series of wind-driven events while using different eddy viscosities to simulate the varying randomness effects. The steady-state models have been used extensively for studying currents in enclosed bodies of water such as lakes and reservoirs. For coastal studies, however, a steady-state analysis may not always be valid.

208. A numerical modeling approach encompassing a spatial extent sufficiently large enough to cover all boundaries pertinent to the evaluation, and with the capability of temporal (time-wise, unsteady) variations, which follows the guidelines of Figures 31 and 32 (Sheng 1983), will result in a comprehensive numerical model that will perform an impact analysis of the effects of disposal of dredged material at an aquatic disposal site. This approach combines the efforts of mathematical modeling with the required verification data resulting from laboratory investigations and in situ prototype data collection and analysis. The numerical model simulations can never be expected to be more accurate than the prototype field data used for their verification; hence, it is important that a sufficient quantity of high-quality verification data be obtained prior to using the numerical model for forecasting efforts. At the same time, collection and analysis of prototype field data are expensive and can be quite time-consuming. These are probably the reasons why more such comprehensive investigations of the ultimate disposition of resuspended material from the mounds at DMADS are not performed. The capability does exist, however, for the application of

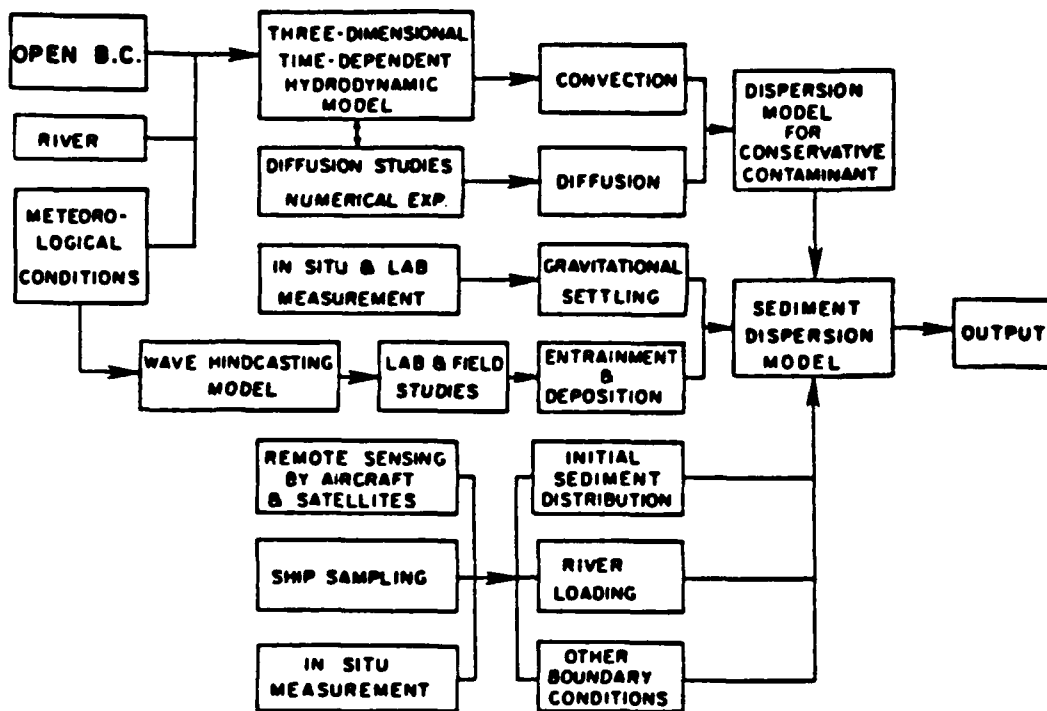


Figure 31. Schematic of comprehensive sediment transport model (after Sheng 1983)

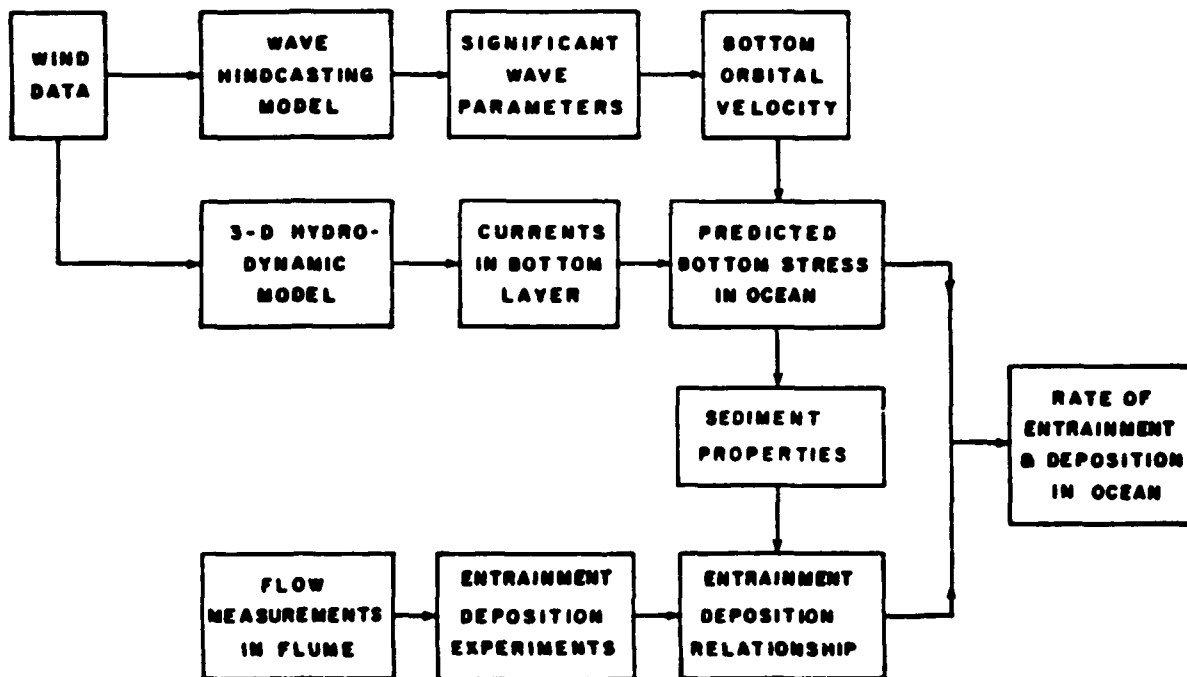


Figure 32. Schematic for defining entrainment and deposition of sediments (after Sheng 1983)

numerical modeling to spatial and time-scale events such as transport and redeposition of dredged material from DMADS.

Field and Laboratory Studies

209. The use of field observations to determine the fate of transported mound material is difficult. The ease of using field observations to determine fate is a function of the type of mound material, the characteristics of the surrounding sediment, and the sophistication of the monitoring program. The depth of deposits is seldom great enough for accurate measurement by standard hydrographic surveying methods unless a definite shoal region develops in the flow field. Sedimentation stakes and deposition pits can be used, but they require either a foreknowledge of where sediment will travel or a rather extensive (expensive) sampling network. An alternative to hydrographic surveying is sediment tracer tests. Either naturally occurring tracers such as mineralogically identifiable grains or artificial tracers such as radio-nuclides can be tracked to detect areas and quantities of deposition. Appendix C provides more detailed discussion of this topic.

210. Small-scale, two-dimensional flumes, or other laboratory tests, have not been developed for the purpose of determining the ultimate disposition of dredged material that has been resuspended from open-water disposal mounds. Three-dimensional physical model studies of a limited nature have been performed for this purpose, however. Such studies were addressed in the section on physical modeling of transport and redeposition (paragraphs 198-203).

PART V: RECOMMENDATIONS

211. Very little guidance exists for the long-term management of DMADS from the standpoint of sediment capacity. The application of numerical sediment transport models to DMADS will yield valuable information for their long-term management. However, direct application of these models to specific sites can be very costly. Because multiyear, continuous simulations are not economically feasible, a capability for providing long-term guidance must be developed. The common technical approach in long-term sediment transport studies is to statistically combine a series of short-term simulations into long-term estimates for a specific site.

212. Many studies do not have sufficient funding to warrant even these short-term simulations at a specific site. Therefore, there is a need for developing a means of economically applying general information to specific sites from a series of short-term numerical simulations of sediment transport at generic sites under a variety of conditions.

213. This development can be accomplished by separating the work into four tasks. The first task should be definition of the range of site characteristics likely to be encountered at a field site. These characteristics should be defined for both open-ocean and estuarine environments. The frequency of occurrence of typical energy levels must be estimated and the range of geometric settings addressed. These parameters include: tidal elevations, tidal currents, storm surge, storm currents, wind magnitude and direction, wave conditions, bottom sediment characteristics, and water depth.

214. A series of generic disposal sites should be developed for sediment transport modeling from the results of the first task. These generic sites must be modeled with a variety of energy sources appropriate for either open-ocean or estuarine environments to provide input for nomograph development. As part of the second task, the program of generic simulations should be designed by identification of significant parameters, nondimensionalization of these parameters, and test simulations to define the range of sensitivity of the results to variation of the parameters. As the generic simulations are completed, development of the nomographs could begin. The development of nomographs from the results of the simulations and the computerization of these techniques would complete the third task.

215. The final task would involve evaluation of the nomograph approach by application to specific field sites and further refinement of the approach. The approach outlined above is probably the most cost-effective, yet technically defensible, means of evaluating erosional characteristics of DMADS.

216. The development of the nomographs and procedures discussed in the preceding paragraphs may require several years. In the interim, steady-state and the time- and rate-dependent analytical methods for estimating erosion and mound size (see Part III) could be further developed. These methods could be adapted for use on microcomputers and eventually added to the ADDAMS system (Hayes et al., in preparation). An instruction report also would be required. Before implementing analytical tools, the approach should be reviewed by other sediment transport experts to try to achieve a general consensus since these methods have not been fully developed or verified.

217. The above developments would address questions dealing with erosion, mound size, and site capacity; however, they would not address questions concerning the transport paths and redeposition (because these processes are strongly dependent on site conditions). To address these issues, the analytical plume models of Part IV could be programmed for use on microcomputers. This could be done easily, and the models would be easy to use. Although these simple models do not include settling of sediment, they would give estimates of plume concentration contours, and deposition patterns could be inferred from this information. Perhaps a better alternative would be to modify the disposal models referenced in Part II to simulate the spread of resuspended sediment in addition to sediment from disposal operations. The disposal models include settling. Physical and numerical modeling are other means of addressing resuspension and redeposition should time and funding permit.

218. Other future research that would provide better means of evaluating the capacity of disposal sites includes the following:

- a. Improved techniques for more accurate measurement of deposition and erosion rates are needed. Present survey methods are never more accurate than 0.5 ft and in actual practice are rarely more accurate than 1 ft. Often these errors are systematic, not random, so that computations of erosion and deposition from surveys can be seriously in error.
- b. Better physical descriptions are needed for the armoring process in sediment beds and for the variation of fine

sediment characteristics with time and stress history. Most of the mathematical relationships describing these processes are strongly empirical and may not represent actual conditions in the field. The improved armoring descriptions would be used to improve predictions made by numerical models and the analytical techniques.

- c. Improved techniques are required for determining the critical shear stress for fine cohesive sediment.
- d. For the limited number of DMADS that are not readily amenable to generic analyses (such as nomographs and analytical tools) or for which the political/economic climate requires a more in-depth study, a site-specific numerical model study should be employed. Although numerical models exist that could be used for modeling DMADS, these models need to be adapted, exercised, improved/updated, verified, and made accessible to the field offices.
- e. Field sites must be monitored to provide data to verify the predictive tools proposed herein.

PART VI: SUMMARY

Purpose of the Study

219. Because of the nature of this report, the summary has been written to stand alone. Thus, the reader can read the summary and obtain the more important information provided in the report. As a result, the summary contains more detail and is longer than usual.

220. In order to manage an open-water dredged material disposal site, it is essential to know the physical capacity of the site: (a) how much material should be dumped in a particular location, and (b) what the capacity of the material to remain onsite is under various environmental conditions of waves and currents. Once dredged material has been placed in open water and has settled to the bottom, it becomes susceptible to erosion, resuspension, and transport away from the disposal area by ambient currents. Understanding the processes responsible for resuspension and transport is critical to site management since the processes, in part, determine the storage capacity of the disposal site. Long-term management of disposal sites requires an understanding of how much area the disposal mound encompasses, when the mound encroaches on the site boundaries, how much material leaves the site, and where the material ultimately goes.

221. The purpose of this report was to identify methods that can be used to develop information concerning the long-term fate of dredged material disposed at aquatic sites. This information will be useful to those involved with the long-term management of DMADS.

222. The methods of analysis presented herein are broken into two major categories: (a) mound resuspension and dynamics, and (b) transport and redeposition of mound material. A variety of methods of analysis could be used for each of these two categories. They range from simple analytical approaches to the most sophisticated numerical modeling techniques. The approach taken here is to present four approaches for each of the two major categories. These are:

- a. Steady-state analytical methods.
- b. Time- and rate-dependent analytical methods.
- c. Modeling (physical, numerical, hybrid).
- d. Field and laboratory studies.

223. The first approach is the easiest to implement, but it yields the least information. This approach assumes steady or constant conditions and would be representative of long-term, average conditions. Such an analysis could fail to show the extreme conditions that can occur during episodic events. The second approach requires more effort to apply than the first, but includes the effects of extreme events by taking into account the frequency of various ambient conditions and the variations in rate-dependent processes. The third approach includes various numerical and physical modeling activities. Modeling can offer the most detail, but it can also require significant effort. Finally, field monitoring of erosion and deposition or laboratory studies, such as flume studies of erosional characteristics, may be useful in themselves, but are generally required to support any serious modeling effort.

224. The types of aquatic sites considered herein are primarily open-water sites, but include oceans, the Great Lakes, and estuaries or tidal waterways. The methods presented were based on established techniques and do not include any new developments. Finally, recommendations are presented for future research and development to improve the capability to predict the long-term fate of dredged material placed in DMADS.

Process Descriptions

225. Resuspension and transport of movable material (cohesive or non-cohesive) is functionally related to the physical stresses (forces) that induce such movement. If the natural environmental forces existing in the region are less than those forces required for particle motion, the material will remain at rest. If, however, the available stresses are greater than those required to place material in motion, motion will be initiated. For deposits containing coarse-grained sediment such as sand, when the environmental forces build up from some value less than that required to cause motion to a value great enough to initiate particle movement, the smaller particles begin to move prior to motion by larger particles. When this happens, the surface area of the disposal mound becomes armored with slightly larger sized material than underlying particles, and the forces (stresses) necessary to initiate motion of this slightly larger material are thereby increased.

226. Thus, for a given intensity of available stresses, the rate of material movement decreases temporally. This is also true of cohesive beds as the critical shear stress increases with depth in the bed (Ariathurai et al. 1977). This implies that the surface area of the mound (the spatial extent) is related to the rates at which material is introduced to the mound by disposal operations and the rate at which sediment is resuspended and transported from the disposal site by hydrodynamic forces and transport processes. If knowledge regarding the magnitudes, duration, and frequencies of the physical forces of the region does not exist, it is essential that a suitable field data-collection program be initiated to obtain those data needed to perform the appropriate analyses.

Stresses that initiate resuspension

227. The transport of sediment or dredged material under the simultaneous action of short-period waves and currents is related to the magnitude of the shear stresses exerted by the combined wave and current motions. The currents that are superposed on the wave-orbital velocities may be tidal currents or other currents of local nature.

228. Tidal and other unidirectional currents. For a control volume concept, the gravitational component of tidal or other unidirectional currents in the direction of flow is balanced by the frictional resistance at the bed, and the resulting equality yields τ_o , the bottom shear stress, expressed as

$$\tau_o = \gamma d S_o \quad (55)$$

where

γ = weight of water per unit volume

d = depth of uniform flow

S = slope of the hydraulic gradient producing flow

229. Defining the friction velocity, u_* , as

$$u_* = \left(\frac{\tau_o}{\rho_w} \right)^{1/2} \quad (56)$$

where ρ_w = density of water. A reference dimensionless quantity incorporating both fluid properties and sediment size, referred to as the shear Reynolds

number, Re_* , can be established using the following:

$$Re_* = \frac{u_* D}{\nu} \quad (57)$$

where

D = particle diameter

ν = kinematic viscosity of water

230. Shields (1936) appears to have been the first in the field of sediment transport mechanics to relate the critical shear stress to Re_* for the special case of uniform sand. The critical shear stress is the bottom shear stress that imitates sediment movement. Since that time, however, other researchers have experimented with a wide assortment of materials with densities ranging from 1.06 to 7.90 g/cm³. These researchers' results are shown in Figure 33, which is a representation of the critical bottom shear stress for sediment movement as a function of the shear Reynolds number. The results for the initiation of noncohesive sediment movement under tidal or other unidirectional currents serve as a quite accurate and general criterion.

231. Other formulas have been used for transport by unidirectional currents, such as the Ackers-White formulas for sand transport. Sand sediment mobility is described by Ackers and White (1973) as the ratio of the appropriate shear force on unit area of the bed to the immersed weight of a layer of grains. The mobility number, F_{gr} , is defined as:

$$F_{gr} = \frac{U_*^n}{\sqrt{gD(S-1)}} \left(\frac{\nu}{\sqrt{32} \log \frac{\alpha d}{D}} \right)^{1-n} \quad (58)$$

where

u_* = shear velocity

n = transition exponent depending on the sediment size

g = acceleration due to gravity

S = mass density of sediment

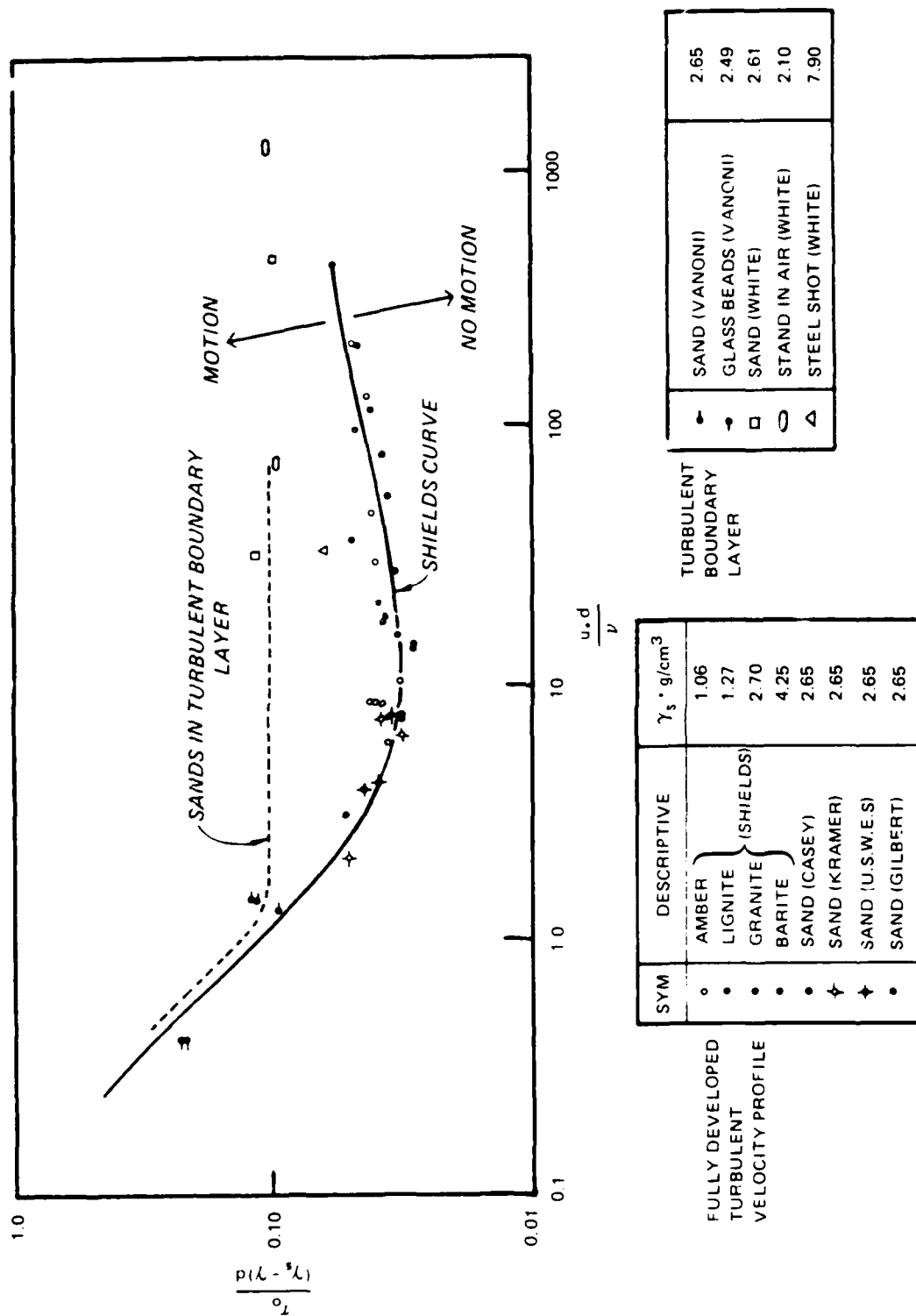


Figure 33. Shields diagram, dimensionless critical shear stress versus shear Reynolds number (after Basco et al. 1974) (Note: γ_s = specific weight of sediment)

V = mean velocity of flow

α = coefficient in rough turbulent flow

d = mean depth of flow

232. A dimensionless sediment transport rate, G_{gr} , is described as

$$G_{gr} = C \left(\frac{F_{gr}}{A} - 1 \right)^m \quad (59)$$

where

C = coefficient in sediment transport function

A = value of F_{gr} at nominal initial motion

m = exponent in sediment transport function

Values of all coefficients and exponents are available in Trawle and Johnson (1986).

233. The sediment in mass flux per unit mass flow rate, X, can be determined from the expression

$$X = \frac{G_{gr} SD}{d} \left(\frac{V}{u_*} \right)^n \quad (60)$$

234. The mass flow rate per unit width, q, is defined as

$$q = \gamma_w VD \quad (61)$$

where γ_w is the unit weight of water. The sand transport by weight per unit width, T, is given as

$$T = Xq \quad (62)$$

235 Quantification of erosion rates of silt-clay sediment is difficult in view of the many variables involved, such as the chemical characteristics of the material, degree of consolidation, armoring, and physical and chemical properties of the water. An equation based on work by Parthenaides (1962) was developed by Ariathurai and Arnlanadan (1978) and used by Trawle and Johnson (1986) to estimate the subsequent erosion and resuspension by unidirectional

currents of clay and silt which settled to the bottom. The Ariathurai equation is

$$\dot{S} = M \left[\frac{\tau_B}{\tau_c} - 1 \right] \quad (63)$$

where

\dot{S} = erosion rate

M = erosion rate constant

= 0.002 kg/sec/sq m for unconsolidated material

= 0.0005 kg/sec/sq m for clumps

τ_B = bed shear stress

τ_c = critical shear stress for erosion

236. The bed shear stress is given as

$$\tau_B = \frac{\rho g N^2 V^2}{d^{1/3}} \quad (64)$$

where

ρ = fluid density

N = Manning's friction factor

Typical representative values of all coefficients, and example computations, are presented by Trawle and Johnson (1986).

237. Wave-orbital velocities. The ability of water waves to transport bottom sediments is related to the magnitude of the shear stress exerted by the wave motion on the bed. Oscillatory fluid motion associated with surface gravity waves exerts shear stresses on the bottom that are often several times larger than shear stresses produced by unidirectional currents of the same magnitude (because of the pressure fluctuations). Thus, the importance of wave motion in initiating and transporting sediments in a coastal environment is apparent. Shear stresses produced by wave motion may put sediments into suspension where they can be transported by currents of a magnitude insufficient to initiate sediment motion.

238. Since the ocean bottom is normally rippled in the areas of coastal engineering interest, any incipient motion criterion should extend from flat

bed to ripple bed conditions. The parameters that were chosen for a graphical presentation by Lenhoff (1982) are a shear Reynolds number, R_* , and a dimensionless grain diameter, D_* , where:

$$R_* = \frac{u_* D}{\nu} 50 \quad (65)$$

and

$$D_* = \left(\frac{\Delta_s g}{\nu^2} \right)^{1/3} D_{50} \quad (66)$$

with

$$u_* = \left(\frac{\tau}{\rho_w} \right)^{1/2} \quad (67)$$

and

$$\tau = \frac{1}{4} \rho_w f_w U_o^2 \quad (68)$$

where

$$\Delta_s = (\rho_s - \rho_w) / \rho_w$$

ρ_s = sediment density

ρ_w = water density

D_{50} = mean grain size

u_* = shear velocity

τ = mean shear stress

f_w = wave friction factor

U_o = maximum orbital velocity for waves at the bed (see Eq. 77)

239. The wave friction factor can be obtained from an empirical relationship of

$$f_w = f\left(\frac{a_o}{r}, RE\right) \quad (69)$$

where

f = functional relationship

a_o = maximum orbital excursion from mean position by motion of particles at the bed

r = bed roughness

RE = orbital Reynolds number

240. The generalized procedure for determining when resuspension occurs due to wave orbital currents may be summarized as follows:

- a. Computation of wave properties U_o and a_o . (U_o is obtained from Equation 77; $a_o = U_o T / \pi$, where T is the wave period.)
- b. Computation of the bed roughness, r , from the following:

$$r = 2D_{90} \text{ for a flat bed (Kamphuis 1975)} \quad (70)$$

$$r = \frac{25(\Delta_r)^2}{\lambda_r} \text{ for a rippled bed (Swart 1976)} \quad (71)$$

where

D_{90} = grain size for which 90 percent of material is finer

Δ_r = ripple height

λ_r = ripple length

c. Computation of a_o/r .

d. Computation of orbital Reynolds number:

$$RE = \frac{U_o a_o}{\nu} \quad (72)$$

e. Determination of wave friction factor, f_w , using empirical relationships based on more than 600 data sets, Figure 34, (Riedel et al. 1972).

f. Calculation of R_* and D_* using Equations 65-68.

241. A total number of 643 data points consisting of combinations of R_* and D_* at the onset of movement were calculated by Lenhoff (1982) using the procedure described in paragraph. The empirical curve for the onset of movement was found to be very closely simulated by the parabolic equation

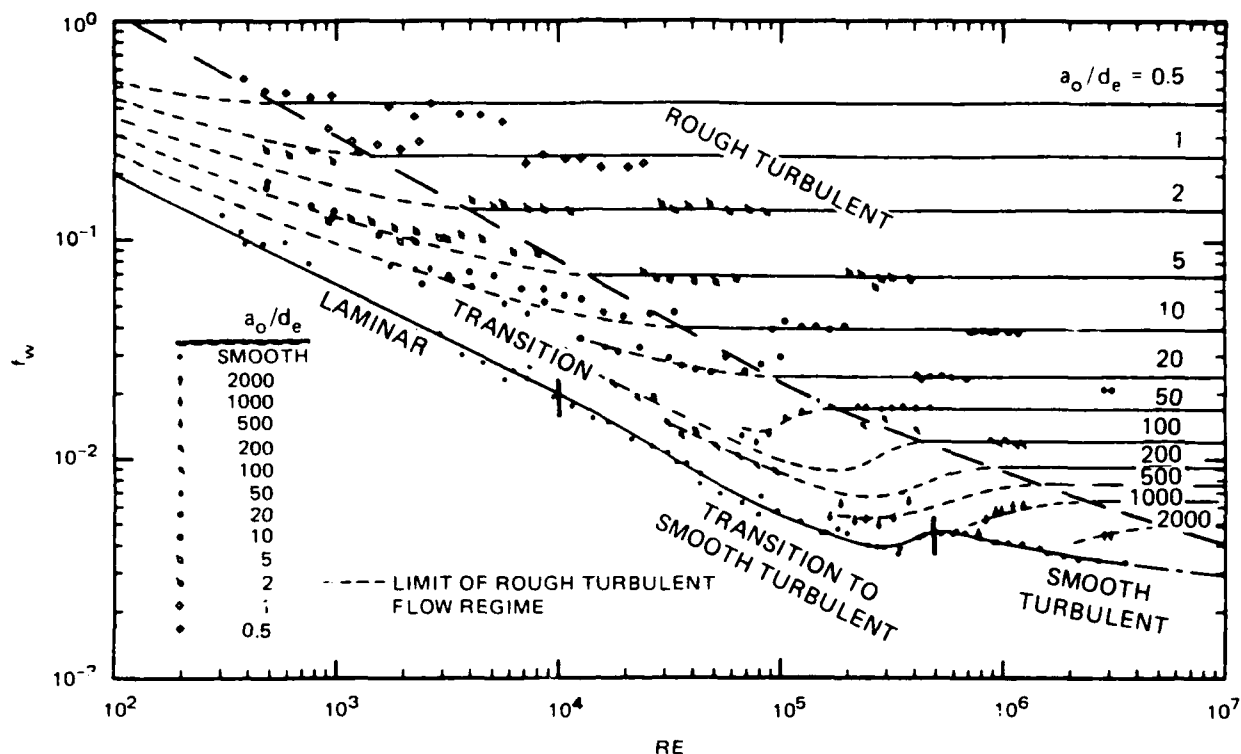


Figure 34. Experimental wave friction factor, f_w (after Riedel et al. 1972)

$$\log_{10} R_* = 0.092 (\log_{10} D_*)^2 + 1.158 \log_{10} D_* - 0.367 \quad (73)$$

Thus, if R_* from Equation 65 is greater than R_* from Equation 73, resuspension should occur.

Combined tidal and wave-orbital currents

242. The conditions for movement under unidirectional steady flow have been widely studied by numerous researchers, and well-established criteria for the beginning of movement exist. The criteria based on work by Shields (1936) and by Ackers and White (1973) are probably the two most widely used and acknowledged in this regard. A direct comparison between these criteria and the results of the study by Lenhoff (1982) can be made. Figure 35 shows the original Shields (1936) and Ackers and White (1973) data points, together with the curve defined by Equation 73. The envelope encompasses data scatter for oscillatory flow.

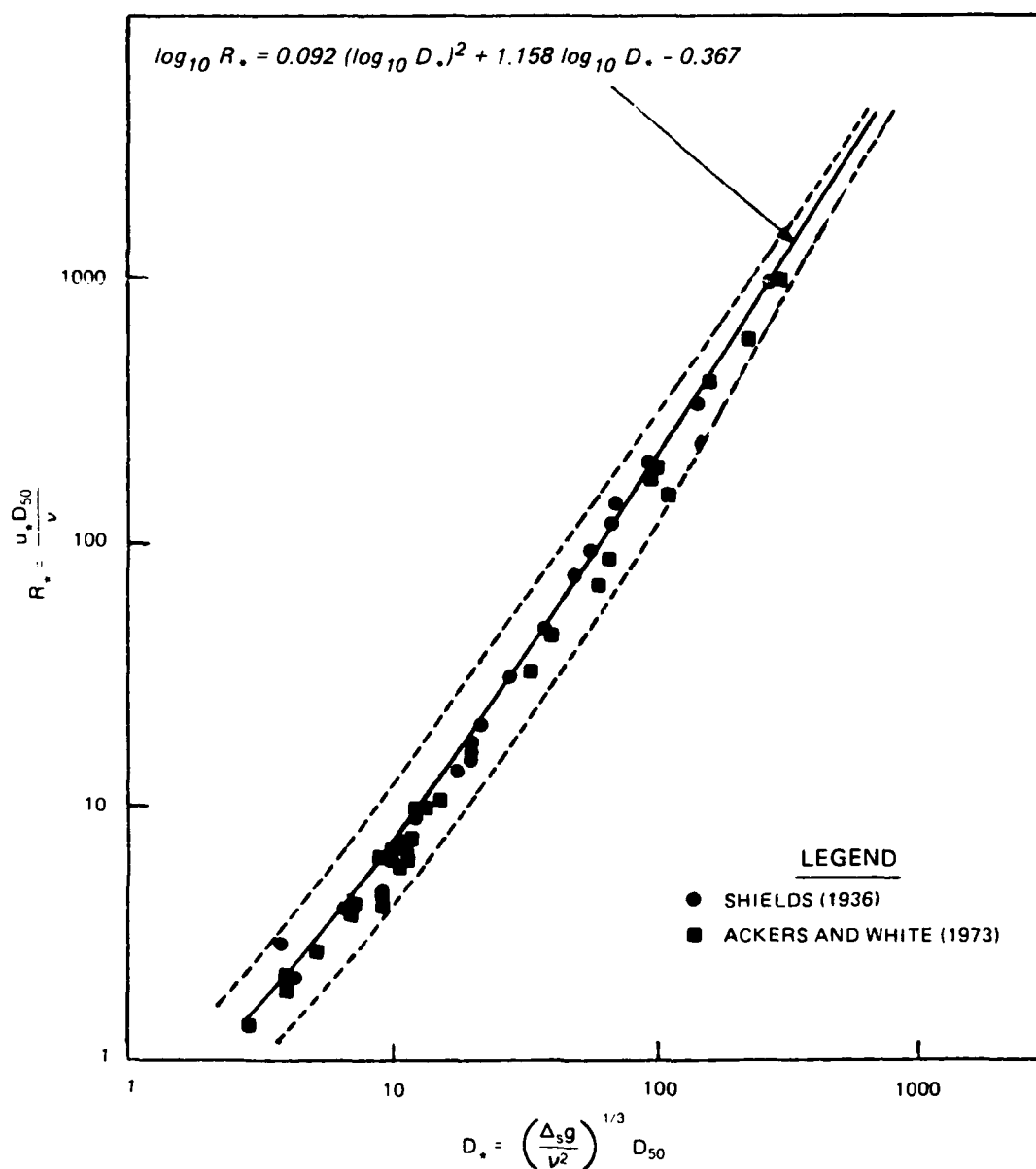


Figure 35. Comparison of incipient motion of sediment particles under oscillatory flow criteria with unidirectional flow criteria (after Lenhoff 1982)

243. The unidirectional data can be seen to fall well within the scatter of the oscillatory flow data points. It can, therefore, be concluded that there is no significant difference between the criteria for uniform flow and the criteria of oscillatory flow as derived in this study. A single criterion can be applied with the same degree of accuracy for both flow conditions. Consequently, it is reasonable to assume that the criterion developed by Lenhoff (1982) can be applied for wave-generated flows,

current-dominated flows, and all combined flow modes in between, provided the appropriate shear velocity, U_* , is used in Equation 65.

244. Another method referred to as the modified Shields curve has been used to determine the threshold of sediment motion due to combined currents and oscillatory waves. Sleath (1978) displayed data previously analyzed by Komar and Miller (1974) by computing a nondimensional grain size, D_* , for a particular grain-size diameter, D , as below:

$$D_* = \left[\frac{(\rho_s - \rho) g}{\rho v^2} \right]^{1/3} D \quad (74)$$

D_* is functionally related to a critical dimensionless shear stress parameter, ψ_c , defined as

$$\psi_c = \frac{\tau_c}{(\rho_s - \rho) g D} \quad (75)$$

This relationship is displayed in Figure 36.

245. For a specific grain-size diameter in the bed, the dimensionless grain size is determined from Equation 74; the dimensionless critical bed shear stress is determined from Figure 35. By knowing the dimensionless critical bed shear stress parameter, the critical bed shear stress for material motion, τ_c , is calculated from Equation 75. Hence; it can be readily determined whether the stresses are great enough to cause material to move by comparing τ_b with τ_c .

246. The bed shear stress, τ_b , for a combined tidal current and wave-orbital velocity can be computed from a Bijker-type equation (Thomas and McAnally 1985) for the shear velocity, u_* , defined as

$$u_* = \sqrt{\frac{1}{2} f_c U^2 + \frac{1}{4} f_w U_o^2} \quad (76)$$

where

f_c = shear stress coefficient for currents, 0.003 (Sternberg 1972)

U = current velocity

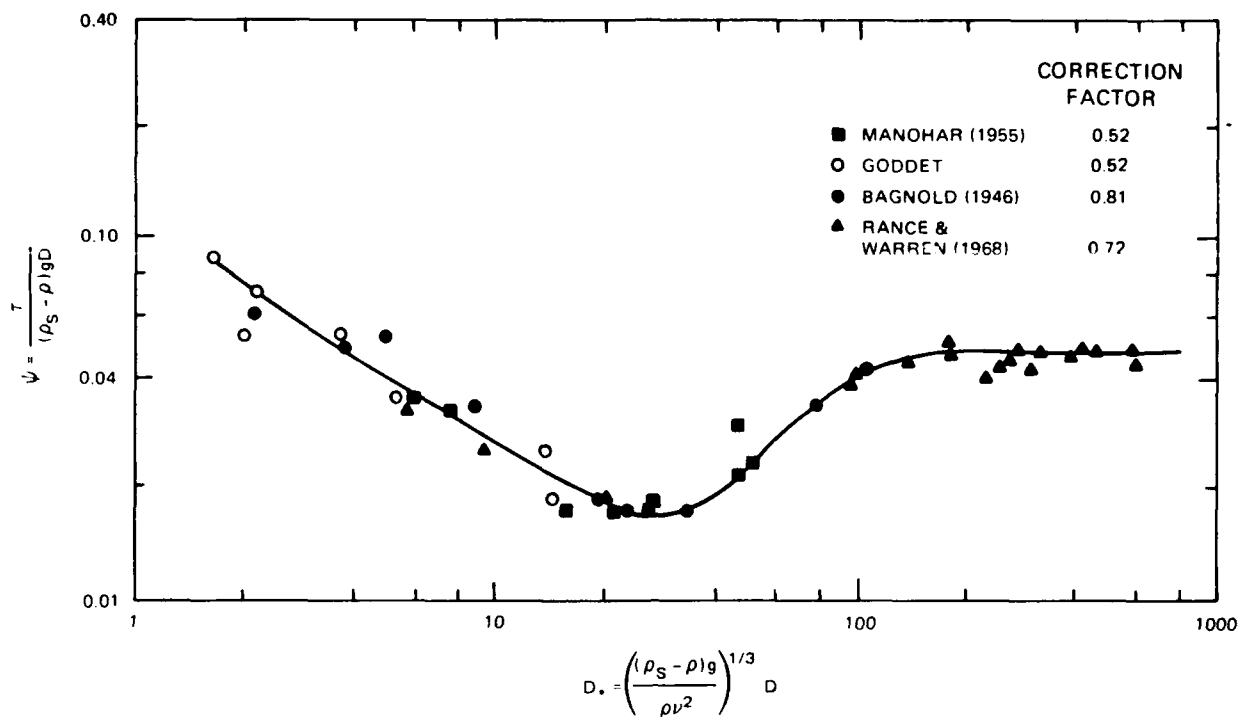


Figure 36. Modified Shields curve for threshold of sediment motion under oscillatory wave motion (after Sleath 1978)

f_w = shear stress coefficient for waves (wave friction factor)

U_o = maximum orbital velocity for waves at bottom

247. The maximum orbital velocity for waves at the bottom can be determined from

$$U_o = \frac{\frac{\pi H}{T}}{\sinh \left(\frac{2\pi d}{L} \right)} \quad (77)$$

where

H = wave height

T = wave period

L = wave length (obtained by iteration of Equation A11 (Appendix A) or published wave tables)

d = water depth

248. The bed shear stress, τ_b , is related to the shear velocity, U_* , as

$$\tau_b = \rho U_*^2 \quad (78)$$

where ρ = density of water .

Determination of wind-wave characteristics

249. The water-wave contribution to sediment transport along the bottom is directly related to characteristics of the wave climate, such as wave height, H , wave period, T , and wave direction of propagation. Prior to the early 1980s, these characteristics of surface gravity waves on water were not generally well known. Some specific locations along the coastline of the United States had been gauged with wave-measuring instrumentation for limited periods of time, but such direct measurement of water surface time-histories was minimal. Wave hindcasting from synoptic weather charts had been conducted for some coastal regions, but these hindcasting efforts were either of very short record duration (usually 3 years of weather records) or the methodology suffered from extensive numerical generation shortcomings, including the application of a singular wave model for the determination of wave statistics.

250. Most knowledgeable researchers agree that the spectral approach is significantly better and, indeed, the US Army Engineer Waterways Experiment Station (WES) initiated a study in late 1976 to provide, through hindcasting, a directional spectral wave climatology for all continental US coastlines (Atlantic, Pacific, Gulf of Mexico, and Great Lakes). This climatological information is being produced by a Wave Information Study that will generate numerical simulation of wave growth, propagation, and decay under historical wind fields of 20-year duration (1956-1975). The final product of this wave hindcasting system is a voluminous data base of wave-parameter data organized by site and time interval. To provide access to this data base for Corps field offices, a computer-based system for storage, retrieval, and computation is operational: the Sea-State Engineering Analysis System (Ragsdale 1983).

Armoring effect on material movement

251. As movable particles are placed in motion, removed from the bed, and transported from the region, the grain-size distribution of the material remaining in the bed surface layer increases, and the time-rate of erosion

decreases. An ultimate depth of scour will be reached for a particular flow intensity because the removal of movable particles results in a layer of particles that are too large to be moved by that particular flow strength. The US Army Engineer Hydrologic Engineering Center (1977) described a method to obtain the depth of scour that occurs before an effective armor layer is fully developed as

$$D_{se} = \frac{2}{3} \frac{D}{PC} \quad (79)$$

where

D_{se} = depth of scour to equilibrium layer

D = largest particle diameter scoured

PC = percent larger than D , as a fraction

252. When the particle critical stress is exceeded, entrainment is considered to be instantaneous for that size material and all finer particles, and these particles are removed from the DMADS.

Mound dynamics

253. Three important questions concerning mound dynamics must be answered for site capacity management of DMADS: how do the mound volume, height, and planar extent vary with time? The specific characteristics of the disposal operations and the erosional characteristics of the mound will determine the answers to these questions.

254. Likewise, the mound size and shape could impact the erosional characteristics. These interrelated effects are difficult to account for. If it can be assumed that the mound size has an insignificant influence on the erosional characteristics and that the shape does not change with time, the problem becomes more amenable to analytical solutions.

Transport and redeposition of mound material

255. When DMADS computations indicate that sediment will erode from the mound, the destination of that sediment is of interest. The transport of resuspended dredged material from a disposal mound and the ultimate redeposition of that material from the water column is governed by an extension of the same physical stresses and forces that initially caused the resuspension from the mound. Redeposition of mound material occurs when the supply of

deposable sediment exceeds the flow's transport capability; thus, a region in sedimentation equilibrium is one in which the flow is just able to prevent quasi-permanent deposition of the available supply of sediment. A change in deposition behavior at a site can be the result of a change in deposable sediment and/or a change in transport capability.

Steady-State Analytical Methods

Mound resuspension and dynamics

256. Steady-state analytical methods for the analysis of mound resuspension and dynamics are solutions of mathematical expressions that can be solved in closed form when ambient parameters of currents, waves, disposed material, and disposal operations can be reduced to essentially constant conditions. Steady-state analyses require: selection and combination of representative energy levels of currents and waves; description of sediment characteristics and disposal mound configuration; and assumptions regarding disposal procedures.

257. The procedure of George and Walton (1984) for estimating the resuspension potential for coarse-grained sediment at the deposited dredged material mound involves a comparison of the total bottom velocity with the threshold velocity for the known grain-size distribution. The total maximum bottom velocity is estimated as the summation of the mean net bottom drift, the maximum tidal velocity, and the maximum wave-induced bottom current:

$$U_{tot} = U_{bm} + U_t + U_w \quad (80)$$

where the linear concept of combining energy levels of waves and currents is considered appropriate. By knowing the typical grain diameters, a threshold velocity, V_t , can be obtained from Shields (1936) (or other data sources), above which erosion of disposed material can be expected. If $U_{tot} > V_t$, the site will tend to be dispersive; if $U_{tot} < V_t$, the site will tend to retain the disposed dredged material.

258. Applied Science Associates, Inc. (1983), developed relationships for a volumetric sediment transport rate on a unit width basis as a function of the mean net bottom drift velocity, U_{bm} , and various wave-induced bottom currents, U_w . A sample of these data displays is presented in Figure 37.

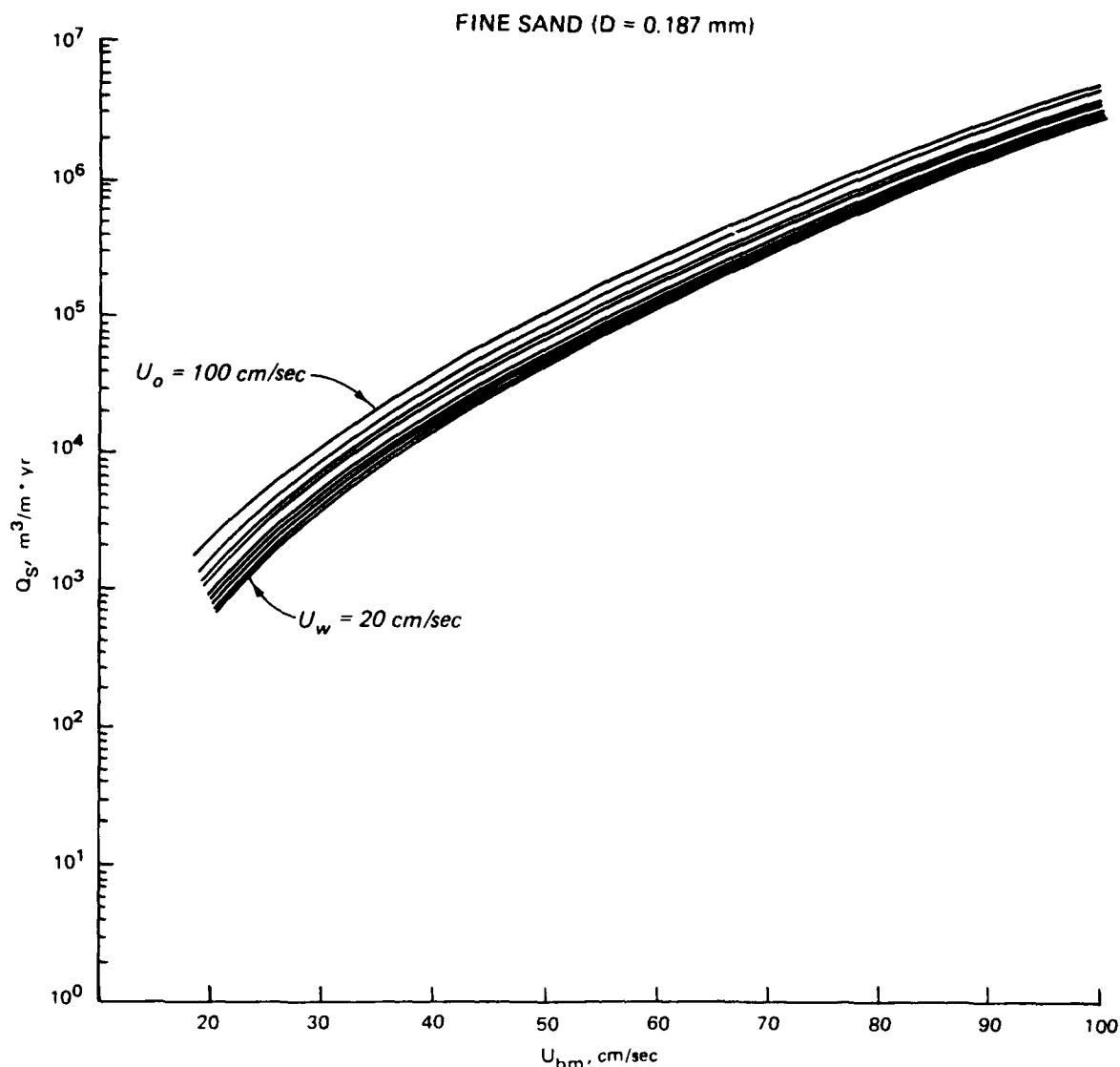


Figure 37. Volumetric sediment transport rate, Q_s , versus mean net bottom drift velocity, U_{bm} , for various wave-induced bottom currents, U_w (after Applied Science Associates, Inc. 1983)

By applying these data displays for the appropriate material (sand, silt, clay, etc.), the annual unit sediment transport rate per metre of width, Q_s , for each material constituent considered, can be determined. The annual sediment transport rate (in cubic metres per year) is the annual unit sediment transport rate multiplied by the width of the receiving area.

$$Q_t = Q_s B \quad (81)$$

Here, the spatial extent of the disposal mound configuration is required for estimating the total sediment transport from these unit width evaluations. If the annual sediment transport rate, Q_t , exceeds the annual disposal rate, no long-term mounding of the disposal site will be expected. If the annual disposal rate exceeds the annual sediment transport rate, long-term mounding can be expected.

259. A steady-state analytical method for mound resuspension was developed by Trawle and Johnson (1986) in an investigation of the Alcatraz (San Francisco Bay) aquatic disposal site. The objective was to estimate both the percent of dredged material initially deposited at the disposal site and the percent of deposited material subsequently resuspended and transported away from the disposal site under various hydrodynamic conditions. The investigation did not include the long-term fate (redisposition) of mound material that leaves the disposal site. This analytical procedure included the use of the Ackers-White transport function for sand transport and the Ariathurai equation for the erosion and transport of clay and silt. Tidal currents of about 3 fps were required for sand movement to begin at this location. Trawl and Johnson determined that current velocities in excess of 1 fps were required for significant erosion of unconsolidated fine-grained material. Current velocities in excess of 5 fps were required to cause perceptible motion of the consolidated materials.

Transport and redeposition of mound material

260. Analytical techniques for predicting the long-term fate of suspended sediment are extensions of the procedures used to determine the amount of sediment that leaves the disposal site. Knowledge of hydrodynamic conditions at the site can be used to calculate directions and rates of transport out of the site. Hydrodynamic conditions along the projected path must be postulated so that zones of deposition can be identified. In tidal zones, sediment may deposit temporarily when currents are at or near slack and be resuspended again during strength of flow, moving in and out of the disposal site with the strength of the tidal flow. Describing this process analytically requires a more detailed knowledge of the waterway's hydrodynamic conditions than is usually available for an analytical study. The analytical study is usually reduced to computing a transport path, assuming a sediment

distribution, and concluding that the redeposited sediment thickness is either negligible or significant.

261. No analytical investigations exist in the technical literature that were conducted explicitly for the purpose of determining the ultimate disposition of material that had been resuspended and transported from a DMADS. However, the concept is readily establishable.

262. Steady-state evaluations depend on comparisons of the stresses generated by hydrodynamic processes with the required stresses for transport at successive cross sections along the flow paths. As the flow intensity decreases, the larger particles will settle from the water column. Successive decreases in flow velocity and turbulence parameters will ultimately permit the settling of all but the finest particle sizes. A priori knowledge of the volume of material available for transport and the sediment grain-size distribution of the material will permit an estimation of the thickness of the accumulated material at successive cross sections. It is logical to assume that a uniform distribution of material thickness will exist between computation sections if discontinuities or otherwise abrupt variations in the flow field are ignored.

263. Both steady-state ebb and steady-state flood currents should be evaluated in an analytical evaluation, with the understanding that any resulting deposition will be the net effect of a long-term average condition. In reality, under prototype flow conditions, the deposited sediment distribution should asymptotically decline in a spatial direction until other currents are encountered that will reduce the rate of deposition or preclude such deposition entirely. The conceptual process described herein has been schematized by Morton and Robertson (1983) in Figure 38.

264. Long-term transport contours have been calculated and mapped by George and Walton (1984) to illustrate the far-field dilution associated with unit concentrations of dredged material sources under conditions of ambient advection and dispersion. The procedure is to use closed-form, two-dimensional (2-D) horizontal solutions of the steady-state mass transport equation, for both point and distributed sources. If the mean net surface drift is less than the mean magnitude of the tidal current parallel to it (x-direction) over half of the tidal cycle, then overlapping of various plumes might be expected and a continuous (steady-state) release plume model should be used. (See following section, "Time- and Rate-Dependent Analytical

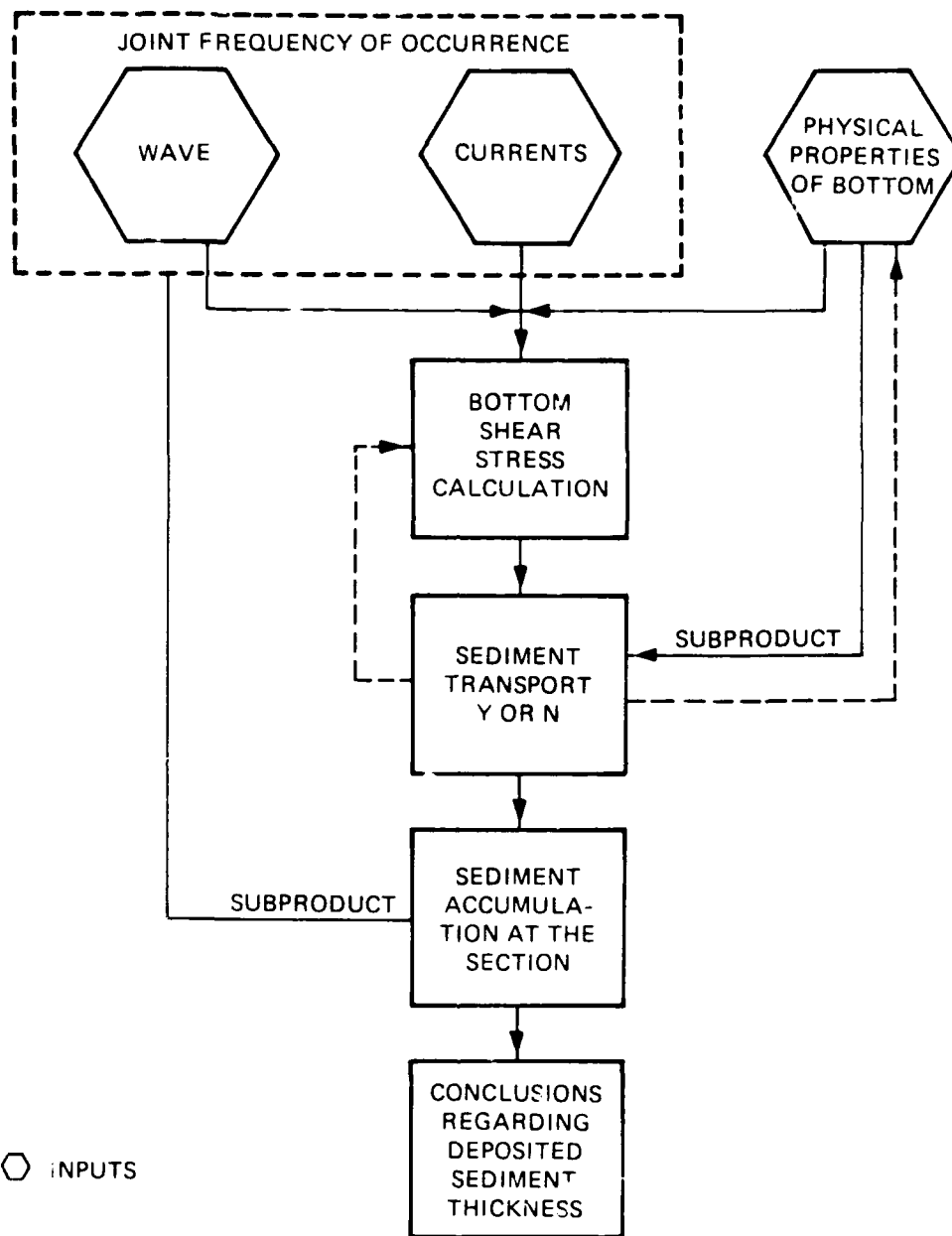


Figure 38. Schematic analysis of transport and redeposition of mound material (after Morton and Robertson 1983)

Methods.") Otherwise, a discrete (unsteady) source plume model should be used. To estimate dispersion coefficients in the x- and y-directions, the tidal excursion in these directions is required.

$$L_x = U_{tx} * \frac{T}{2} * \frac{\pi}{4} \quad (82)$$

and

$$L_y = U_{ty} * \frac{T}{2} * \frac{\pi}{4} \quad (83)$$

where

L_x = tidal excursion parallel to mean net surface drift

U_{tx} = maximum tidal velocity parallel to mean net surface drift

T = tidal period

L_y = tidal excursion perpendicular to mean net surface drift

U_{ty} = maximum tidal velocity perpendicular to mean net surface drift

265. Using the tidal excursion lengths, L_x and L_y , dispersion coefficients can be estimated in the x- and y-directions, using a conservative simplification of the so-called "four-thirds" rule.

$$D_x = 0.0018 * L_x \quad (84)$$

and

$$D_y = 0.0018 * L_y \quad (85)$$

where D_x and D_y are dispersion coefficients parallel and perpendicular to the mean net surface drift direction, respectively, in square metres per second.

266. If the mean net surface drift, U_m , is less than the mean tidal velocity ($U_{tx} * \pi/4$), the site is assumed to behave as a continuous point source under successive disposals or during mound erosion. In this case, a 2-D continuous point source plume model with unit loading ($q = 1$) should be used to develop characteristic concentration contours:

$$C(x,y) = \left[\frac{q}{d * (4\pi * D_y * U_m * x)^{1/2}} \right] * \exp \left[- (y^2) * \frac{U_m}{4 * D_y * x} \right] \quad (86)$$

where

$C(x,y)$ = concentration at coordinate (x,y) , with mound at $(0,0)$

q = mass input rate/sec (= 1 for unit contours)

d = calculation depth, m ($d = h_p$ if above the pycnocline,
 $d = h - h_p$ if below the pycnocline, or $d = h$ if entire water
depth is used)

h_p = pycnocline depth, m

h = mean water depth, m

U_m = mean net surface drift, m/sec

267. As discussed, the disposed material will accumulate on the bottom if the threshold velocity for material movement is greater than the total maximum bottom water velocity (paragraph 260). During those portions of the tidal cycle when slack-water conditions exist (e.g., when the tidal current is reversing from flood to ebb, or vice versa), the total maximum bottom fluid velocity may become too small to continue moving material that is in transport. In this case, material that was in transport may deposit on the bottom. As the tidal velocity increases in the opposite direction, the material may again be placed in suspension and carried in a different direction. The ultimate location of the material will be determined by the divergence of the flow field with respect to the net transport capacity for each portion of the tidal cycle. The potential for resuspension exists when the total maximum bottom fluid velocity exceeds the threshold velocity for material movement.

268. The 2-D, continuous point source plume model provides the dilution contours under steady-state current conditions (average values). When the resuspended material is transported to a region of the flow field where conditions of current velocity become less than that required for suspensions, the material being transported in the suspension plume will settle from the water column to the bed, where it will remain until stresses again become sufficient for resuspension and transport to another location.

Time- and Rate-Dependent Analytical Methods

Mound resuspension and dynamics

269. Turbulent flow and wave motions are, in the strictest sense, unsteady. However, if average values are taken over sufficiently long periods of time, the resulting motions and accelerations may be evaluated on a steady-state basis. When it is determined that more detailed information is required than that available through the averaging process, it becomes necessary to treat the time- and rate-dependent physical processes in an unsteady manner.

270. Most of the physical phenomena associated with sediment movement and energy sources that cause such movement are continuous. The computing systems designed to process simulations of these physical processes are discrete. Hence, it becomes necessary to discretize the continuous processes into a series of distinct elements, to select those discrete values of the various energy sources at the appropriate time, and to simulate the processes as a definite series of time events or unit steps of the rate changes that may be occurring.

271. The steady-state analytical method for evaluating mound resuspension at the Alcatraz aquatic disposal site, developed by Trawle and Johnson (1986), was extended to cover time- and rate-dependent conditions as well. This analysis was a time-increment application of the Ackers and White (1973) transport formula that estimated the capability of the ambient currents to remove sand from the dump site and the Partheniades (1962) erosional equation that estimated the resuspension of clay and silt. A time-dependent analysis was conducted by summing transport over a tidal cycle.

Transport and redeposition of mound material

272. There are no reports in the literature of time- and rate-dependent analytical investigations regarding the transport and redeposition of resuspended disposal mound material expressly for the purpose of ascertaining its ultimate fate. However, time and rate analyses are extensions of the steady-state considerations under the added stipulations that velocities may vary with time at the particular locations of interest. Also inherent in the analysis is the added assumption that additional material may be placed in the disposal area with time. Thus, the time- and rate-dependent analyses become incremental calculations of deposition potential and accumulation at specific

cross sections in the flow field at finite increments of time, during which the current velocities may have varied and the sediment concentrations in the water column may have been altered.

273. Under these considerations, the material previously deposited at a section under low current velocities may be again resuspended and transported to another section in the flow field. Hence, an accounting must be performed of the conditions at each section. Such analytical methods should be computerized for rapid data processing. Therefore, this approach would have much resemblance to numerical modeling, and again, modeling is the preferred approach.

274. The work of George and Walton (1984) was extended to investigate time- and rate-dependent analyses of far-field dilution associated with 2-D, discrete point sources and with 2-D, discrete distributed source models. The procedure is to provide closed-form, 2-D, horizontal solutions of the transport equation for both point and distributed sources. Redeposition of the suspended material being transported by the plume occurs when the stresses determined by the total bottom velocity components become less than the threshold values for material transport. Thus, settling could be only crudely accounted for.

275. If the mean net surface drift, U_m , is greater than the mean tidal velocity, $(U_{tx} * \pi/4)$, the site is assumed to behave as a series of discrete loadings that never merge. In this model, the source is also assumed to be small enough to be considered a point source when viewed from the far field. The following 2-D equation describes the horizontal movement and dispersion of the plume through time:

$$C(x,y,t) = \frac{Q}{4\pi * d * \left(D_x * D_y\right)^{1/2} * t} \quad (87)$$

$$* \exp \left[\frac{-(x - U_m * t)^2}{4 * D_x * t} - \frac{y^2}{4 * D_y * t} \right]$$

where

$C(x,y,t)$ = concentration at coordinates (x,y) at time t , with $(0,0,0)$ as the point and time of disposal

Q = mass unit (= 1 for unit contours)

D_x, D_y = dispersion coefficients parallel and perpendicular to the mean net surface drift direction, respectively, sq m/sec

Concentration contours are developed by calculating and plotting on a rectangular grid the concentrations for several sets of (x,y) coordinates at different times, t . The concentration contours are formed by interpolating between the calculated values for lines of equal concentration.

276. If the mean net surface drift, U_m , is greater than the mean tidal velocity and if the source is assumed to have finite dimensions (distributed by a moving dump), the following 2-D equation describes the horizontal movement and dispersion through time (George and Walton 1984).

$$C(x,y,t) = \frac{C_o}{4} * \left\{ \operatorname{erf} \left[\frac{\frac{b}{2} - x + U_m * t}{4 * D_x * t^{1/2}} \right] + \operatorname{erf} \left[\frac{\frac{b}{2} + x - U_m * t}{4 * D_x * t^{1/2}} \right] \right\} * \left\{ \operatorname{erf} \left[\frac{\frac{a}{2} - y}{4 * D_y * t^{1/2}} \right] + \operatorname{erf} \left[\frac{\frac{a}{2} + y}{4 * D_y * t^{1/2}} \right] \right\} \quad (88)$$

where

C_o = initial concentration (= 1 for unit contours)

erf = error function

$b = v * t'$ (initial plume dimension along the x-axis), m

v = disposal vessel speed, m/sec

t' = individual disposal duration, sec

$a = 2 * w$ (initial plume dimension along the y-axis), m

w = disposal vessel width, m

Modeling

Physical modeling

277. Physical hydraulic models are scaled representations of specific water bodies and their associated boundaries. The behavior of flows in the scale model is related to behavior of flows in the prototype that the model represents, by means of scaling relationships. Saltwater supply, tide

generators, wave generators, and gauged freshwater inflows are necessary appurtenances. The models are often molded in concrete between closely spaced templates, although many coastal and estuarine models are constructed with movable-bed boundaries. Instrumentation is used on the models to measure such attributes as water surface elevation, current speed and direction, salinity, and tracer concentrations. Water surface tracers and dye patterns are often photographed to qualitatively and quantitatively examine patterns of flow in the model.

278. For coastal and estuarine studies not concerned with the movement of sediment, fixed-bed models can often be developed to provide kinematic and dynamic responses that are indicative of the prototype conditions. Specifically, fixed-bed models provide information regarding velocities, discharges, flow patterns, water surface elevations, and energy losses between points in the prototype. An undistorted-scale model ideally provides greater insight, at less effort, into the refraction and diffraction phenomena associated with waves passing underwater topography and around coastal features. Accordingly, a fixed-bed, undistorted-scale model can be effectively used for the analysis of kinematic and dynamic conditions associated with waves, current intensities and patterns, discharges, and forces existing along coasts and in bays or estuaries.

279. Physical hydraulic coastal and estuarine models are frequently distorted for various reasons. Many regions of interest are large, and the flood and ebb tidal deltas may be quite shallow, leading to large model energy attenuation and viscous friction scale effects on waves. These effects can be reduced through distortion of the vertical and horizontal length scales while at the same time decreasing model costs. Reproduction of the entire tidal estuary in the model is often desirable since inclusion of the tidal estuary results in the flexibility to study the effects of improvements on the tidal prism, tidal circulation, tidal flushing, and salinity of the estuary.

280. The movement of loose bed material is governed by the inertial forces of the particles and of the water against them, by the weight of the particles, and by the viscous forces acting between the water and the particles. Three physical laws have evolved from an analysis of these forces: Newton's law of inertia; the law of gravitation; and the viscous friction law of Newtonian fluids. These laws provide the well-known dimensionless terms that must be equated between the model and the prototype for kinematic and

dynamic similarity to prevail, i.e., the Reynolds number and the Froude number. The simultaneous conformation of the model and prototype to both the Reynolds number and Froude number yields the familiar problem that the length-scale factor becomes a function of the scale factor of the kinematic viscosity. This means that no readily available fluid possesses the kinematic viscosity to make a useful model fluid.

281. For the application of strictly coastal sediment modeling problems, Migniot et al. (1975) stated that, since all of the similitude conditions involved cannot be satisfied, the model scales, the material size and density, and the current exaggeration cannot be determined by straightforward computations but must be chosen to obtain the most favorable balance between all relevant phenomena. In many respects, this is more an art than a science, and a feeling for the problem, previous experience, and a perspective of the relative importance of each factor are of paramount value. Sedimentation verification in movable-bed physical models is based on prototype observations and is accomplished by selecting an appropriate model sediment and developing the necessary model-operating technique to reproduce the observed scour and fill patterns.

282. Complete three-dimensional physical modeling of the movable-bed aspects of DMADS is practically nonexistent. Apparently, the only such experimental investigation ever conducted and reported in the literature was performed by WES (Simmons and Boland 1969). This completely movable-bed model study of a disposal area was performed as a phase of the three-dimensional comprehensive model study of the Galveston Harbor entrance, Texas. This portion of the overall study was performed to investigate the possibility of disposed dredged material reentering the navigation channel near the entrance to Galveston Harbor. The movable-bed model material was scaled-dimension crushed coal subjected to model testing procedures that forced the crushed-coal bed to respond in geometry as the prototype sand-silt-clay mixtures had evolved. This is a trial-and-error verification procedure that answers most questions regarding shoaling and erosion of navigation channels, but may not satisfy all relationships involved in DMADS dynamics.

Numerical modeling

283. Numerical models are descriptive mathematical expressions that are solved by computational methods. The computational methods of approximation and iteration performed by high-speed digital computers allow solution of

complex equations that do not have closed-form solutions and cannot be solved by analytical methods. The use of numerical models for DMADS can be classified into two groups: models for predicting the short-term behavior of disposed material during and immediately following disposal (see Part II), and models for predicting the long-term fate of the mound material in terms of resuspension, transport, and redeposition. A numerical model study exclusively for the purpose of ascertaining the erosion from a DMADS and the transport and redeposition of mound material has not been conducted at this time. However, numerical models are available that could be applied to DMADS.

284. Two types of numerical models (hydrodynamic and sediment transport) are required to simulate long-term behavior of DMADS. The information derived from hydrodynamic models forms part of the database required by the sediment transport models. Numerical models of hydrodynamic processes and sediment transport are said to be coupled if they are applied simultaneously and interactively on a computer system. If, conversely, the hydrodynamic model is run and the output from it is used as input to the sediment transport model, the two models are said to be uncoupled. In many instances, it is more economical to run uncoupled models. Uncoupled models are unacceptable where sedimentation substantially affects the flow.

285. Two-dimensional, depth-averaged models are most commonly employed in the investigation of tidal flows in inlets, bays, and estuaries. Two distinctly different formulations have been employed and are currently being used at WES: finite difference and finite element. The finite difference model WIFM (WES Implicit Flooding Model) evolved from early work by Leendertse (1967, 1970) and its application have been refined and significantly improved at WES. The model has been described at different stages of development by Butler (1978). The finite element flow model of Research Management Associates (RMA-2) (Norton and King 1977) evolved from work by Norton et al. (1973). The WES version of this model and an uncoupled companion sediment transport model (STUDH), and their application to project studies, have been described by McAnally et al. (1984a, b). A user's manual for these finite element models and support programs (TABS-2) have been prepared by Thomas and McAnally (1985).

286. Breadth-averaged, 2-D models are applicable in studies of relatively deep, narrow channels where lateral currents of appreciable magnitude do not develop. Since few systems meet this criterion, work on models of this

type has been more limited than on the depth-averaged models. However, work performed during the last few years has produced a useful model, LAEM (Laterally Averaged Estuarine Model), by Edinger and Buchak (1981). This model has been used to investigate the effect of navigation channel deepening on salinity intrusion in the Lower Mississippi River and has recently been modified to include sediment transport and applied to the Savannah River Estuary (Johnson, Boyd, and Keulegan, in preparation).

287. Depth- and breadth-averaged 2-D models lack the ability to predict secondary flows involving the dimension that has been averaged. In some instances, these secondary currents may be appreciable and can affect such things as salinity intrusion, sediment transport, thermal distribution, and water quality. Leendertse and Liu (1975) pioneered the development of one of the early 3-D models of an estuary. This model employed Cartesian coordinates. A 3-D model that uses stretched coordinates in both the horizontal and vertical directions has been developed and applied in studies of the Mississippi Sound (Sheng and Butler 1982; Sheng 1983, 1984). This model, CELC3D - (Coastal, Estuarine, and Lake Currents; Three-Dimensional), may be used to provide detailed computations of the currents within several tidal cycles or time scales of a storm event. For a scenario of repeatable hydrodynamics, CELC3D may be combined with sediment transport algorithms for long-term computations on the order of weeks, months, or longer. Three-dimensional versions of the finite element flow and sediment models have also been developed and applied to several field sites (Ariathurai 1982, King 1982). Improvements in the efficiency of computational equipment and modeling technology are increasing the feasibility of applying 3-D models.

288. Coastal processes of tides, waves, wave-induced currents, and sediment transport can be modeled by using the numerical modeling system CIP (Coastal and Inlet Processes). The system utilizes: the WES model WIFM for tides; the Regional Coastal Processes Wave Propagation Model (RCPWAVE) for waves; the model CURRENT for wave-induced currents; and a sediment transport model for transport of sediments due to the combined action of tides, waves, and wave-induced currents. All four models generally use the same computational grid for a given set of conditions.

289. Numerical modeling is the most promising method for evaluating the erosional, transport, and redepositional characteristics of DMADS. Numerical modeling of DMADS will become more practical as computer speed increases.

Hybrid modeling

290. Choosing one of the methods for studying dredged material movement (i.e., field tests, analytical solutions, numerical modeling, and physical modeling) requires that the engineer make balanced trade-offs among time, cost, and accuracy. The choice of a particular model results in some trade-offs, since each model has its own strengths and weaknesses.

291. Field (prototype) data collection/analysis serves both as an important aspect of the basic study methods and as an independent approach. It is an indispensable element in verification of numerical and physical models. To a limited extent, field data can be used to estimate an estuary's response to different conditions of tide and river discharge. Obtaining sufficient temporal and spatial data coverage in the field, however, is a formidable and difficult task. Field testing of structural alternatives costing millions of dollars is far too costly and too risky to pursue.

292. Analytical solutions are those in which answers are obtained by use of mathematical expressions. Analytical models usually combine complex phenomena into coefficients that are determined empirically. The usefulness of analytical solutions declines with increasing complexity of geometry or increasing detail of results desired.

293. Numerical models are capable of simulating some processes that cannot be handled any other way. Numerical models provide much more detailed results than analytical methods and may be more accurate, but they do so at the expense of time and money. They are also limited by the modeler's ability to formulate and accurately solve mathematical expressions that truly represent the physical processes being modeled.

294. Physical scale models have been used for many years to solve coastal hydraulic problems. Physical models of estuaries can reproduce tides, freshwater flows, longshore currents, and 3-D variations in currents, salinity, and pollutant concentration. Conflicts in similitude requirements for the various phenomena usually force the modeler to neglect similitude of some phenomena in order to more accurately reproduce the more dominant processes.

295. Practice in recent years has been to combine two or more of these methods with each method being applied to that portion of the problem for which it is best suited. For example, combining physical modeling with numerical modeling and using field data to define the most important processes and verify the models is termed a "hybrid modeling method." Combining them in a

closely coupled fashion that permits feedback among the models is termed an "integrated hybrid solution." By devising means to integrate several methods, the modeler can include effects of many phenomena that previously were either neglected or poorly modeled, thus improving the accuracy and detail of the results. The hybrid modeling technique is exemplified by the first integrated hybrid study performed, the Columbia River entrance study. A detailed description of that effort is provided by McAnally et al. (1984a,b).

Field and Laboratory Studies

Field studies

296. Field investigations are fundamental and essential aspects in any evaluation of the long-term behavior of dredged material placed in any aquatic environment (riverine, estuarine, lacustrine, coastal, or oceanic). The degree, time, and complexity for conducting such studies can vary through several orders of magnitude. The attributes of interest are similar to those of any other study involving the transport of naturally placed sediment in addition to attributes specifically related to or initiated by the disposal operations (e.g., mudflows, turbidity plumes, consolidation, grain-size equilibrium). In general, field studies are used to complement or verify other methods of study (analytical, laboratory, or modeling) available for predicting the long-term fate of deposited dredged material and to determine the useful life and capacity of a DMADS. Field studies are very expensive to carry out; therefore, the conditions under which the observations are made should be carefully chosen and thoroughly reported.

297. Field studies encompass any prototype-scale experiments made to determine the behavior of disposed dredged material. They can include actual or scaled deposits and may also include studies of the movements of natural or tracer materials. Since the long-term transport and fate of materials are dependent on the short-term behavior (i.e., the manner in which the material is convected and settles to the bed), all time and space scales are appropriate to these studies.

298. Near-field studies are conducted to define the extent and nature of the plume during its descent, the surge and collapse after the material impacts the bed, the turbidity that escapes from the disposal site, the areal

distribution and composition of material on the bed, and the thickness of the deposited material.

299. Long-term field studies are conducted to define the path of turbidity that escapes during the disposal operation and its ultimate fate. They are also conducted to determine resuspension and transport of deposited material from the disposal site and the ultimate fate of the sediment. Long-term studies are conducted to record the stability of the disposal mound and vertical processes such as consolidation or mixing that take place there.

300. Available background data at the site and its vicinity and studies on similar disposal operations should be reviewed before a field study is designed. The objectives of the study should be clearly identified and a reasonable experimental design developed.

301. The behavior of the site under study will probably depend on a number of environmental (physical) factors. These factors should all be considered in the experimental design stage even though they may have little or nothing to do with the actual disposal operation. Arrangements should be made to obtain as much monitoring data as possible on appropriate environmental variables.

302. A data quality-control program should be considered as part of the experiment design. Quality control should include calibration and operational verification on all instruments used. Duplicate, spiked, and standard samples should be used as checks on analytical and sampling methods. Sample custody logs should always be maintained.

303. The study design should be coordinated with other work or studies at the disposal site or in the area. Costs can be reduced if data can be shared. Sample sharing may be possible if sample handling is appropriate to all studies involved.

304. An example of a field study is the Disposal Area Monitoring System (DAMOS) program that was initiated in 1977 by the US Army Engineer Division, New England (1984), to provide scientific data for management and monitoring of dredged material disposal sites throughout New England. Disposed dredged material stability has been monitored by DAMOS since 1977, primarily through bathymetric techniques to evaluate changes in topography which could result from energy levels sufficient to cause erosion and transport of dredged material.

305. The results of the DAMOS surveys have shown conclusively that dredged material at all sites is stable under normal conditions. However, storm events and bioturbation can cause resuspension and transport of material on both large and small scales. None of the New England disposal mounds has given any indication of bed-load transport resulting in a spread of material in the vicinity of the flanks, and it appears that if material is in suspension, it is transported a substantial distance from the site before being redeposited. Consequently, the dilution of material in background sediment loads makes detection of dredged material extremely difficult, if not impossible. All bathymetric surveying techniques have indicated no significant loss of material from the New England disposal sites once disposal is complete and consolidation of deposits is in advanced stages.

306. The use of field observations to determine the fate of transported mound material is exceedingly difficult. The depths of the deposits are seldom large enough for accurate measurement by standard hydrographic surveying methods, unless a definite shoal region develops in the flow field. Sedimentation stakes and deposition pits can be used, but they require either a foreknowledge of where sediment will travel or a rather extensive (expensive) sampling network. Additionally, these methods do not indicate if redeposited sediment originated from the disposal mound or other sources. An alternative to hydrographic surveying is sediment tracer tests. Either naturally occurring tracers, which are mineralogically identifiable, or artificial tracers such as radionuclides can be traced to detect area and quantities of deposition.

Laboratory studies

307. Over the years, a tremendous amount of research in the form of laboratory studies has been conducted on the physical hydraulic aspects of sediment motion. Such studies have been conducted with sand, clay, and silt, with unidirectional currents or scaled-wave climates. A few tests have been performed in the presence of both a unidirectional current and a superposed wave field, although these tests have certain inherent limitations unless the experiments are extremely well designed. Laboratory tests to evaluate the physical characteristics of actual dredged material obtained from open-ocean disposal areas (sand, silt, and clay mixtures) with respect to their resuspension and transport capabilities are even more limited than other tests of disposal site material. The results of such tests, however, provide

information pertinent to site capacity and retention capability and thus have inherent direct application potential.

308. Small-scale 2-D flumes, or other laboratory tests, have not been developed for the purpose of determining the ultimate disposition of dredged material that has been resuspended from open-water disposal mounds. However, 3-D physical model studies of a limited nature have been performed for this purpose.

309. Laboratory tests are carried out to observe some behavior of the sediment disposed at a site or the sediment originating at or near the site. It is necessary to collect representative samples of these sediments and of the native water into which they are discharged. Laboratory tests classify or characterize, provide model input data, and check model behavior.

310. The sediment involved in field or laboratory tests should be well characterized, especially if it is fine-grained material. Characterization will allow the results of these tests to be extended to other situations and can improve the interpretation of the results. Appropriate parameters for characterization should relate to other important behaviors of sediment, such as shear strength and settling velocities. These parameters should be determined for the sediments, suspension water, pore water, and sediment bed structure. For sediments, the following analyses are important: sediment grain-size distribution; organic content; cation exchange capacity; and oil and grease content. For suspension and pore water, the analyses should consider salinity or total filterable solids, sodium absorption ratio, and pH. The sediment bed analyses should include bulk density variation with depth, and liquid limit and percent moisture content.

311. Laboratory studies can be carried out with flows to determine the lowest (highest) bed shear stresses at which erosion (deposition) will occur. The rate of erosion (deposition) is also determined in these tests. Erosion tests on dredged material should be performed on so-called deposited beds, as this state of consolidation is representative of materials that have been dredged, mixed with water, and settled. Erosion tests on consolidated or partially consolidated sediment would use remolded or cored sediments that have relatively low moisture contents.

312. Studies on erosion of newly deposited material and on deposition require a recirculating flume. A suspension is created and the flow is reduced, usually in steps, to allow a bed to form. Visual observations of the

bed should be made, especially if mixtures of cohesive and noncohesive materials are being studied. The flume should have minimal secondary currents, and pumps should not destroy the flocculent structure of the sediment. Usually, concentration in the flow is measured with time, along with the bed shear stress (corrected for sidewall effects). Thicknesses or densities of deposited material are also measured.

313. Apparently, no studies other than that by Moherrek (1978) for the DMADS at Galveston, Tex., have been reported dealing directly with sediment mixtures containing near-equivalent amounts of sand, silt, and clay that have been obtained directly from a DMADS. Two-dimensional, rectangular, circulating flume experiments were performed using four sand/silt/clay mixtures sampled from the disposal site in order to determine the critical erosion velocity, shear stress, and modes of sediment transport for each mixture. Also, an analysis of the hydrodynamic regime for offshore Galveston was performed based on meteorologic and oceanographic data collected between February 1975 and June 1976. Results of the flume experiments and hydrographic analyses were extrapolated to sediment transport processes believed operative in the offshore disposal site.

314. Threshold erosional velocities and rates of erosion were determined by Gularte et al. (1977) for material dredged from the Taunton River, Massachusetts, and the Thames River, Connecticut, prior to being placed in an open-ocean disposal site. Both of the materials tested can be classified as marbled black-olive drab, fairly well-graded silty clay of high organic content. The Taunton River material contained approximately 20 percent by weight clay-size particles (less than 0.002 mm) and 70 percent silt-size particles (0.06 to 0.002 mm), with the remainder fine sand and shell fragments. The material dredged from the Thames River contained much less clay-size particles (10 percent), slightly less silt-size particles (60 percent), and 30 percent sand-size material and shell fragments. The dredged material was tested in a research water tunnel designed and fabricated specifically for this research.

315. Studies of the entrainment rates of descending plumes or jets, the mounding and spreading of material as it impacts the bed, and the limits of bottom spreading can be studied in scale models (tank tests). The sophistication required of the tests and of the tank facility varies greatly depending on the objectives of the tests. Scaling relationships must be carefully

defined. The initial dynamic behavior has been modeled using Froude scaling laws. The ambient receiving waters in the tank can be quiescent or flowing, homogeneous or stratified. The disposal operation can be moving or stationary and can have varying degrees of realism in the portrayal of the vessel. Tank tests should be considered if other methods of analysis are lacking or untested, although the mathematical disposal models discussed in Part II address many of these aspects.

Recommendations

316. Very little guidance exists for the long-term management of DMADS from the standpoint of sediment capacity. The application of numerical sediment transport models to DMADS will yield valuable information for long-term management. However, direct application of these models to specific sites can be very costly. Because multiyear, continuous simulations are not economically feasible, a capability for providing long-term guidance must be developed. The common technical approach in long-term sediment transport studies is to statistically combine a series of short-term simulations into long-term estimates at a specific site.

317. Many studies do not have sufficient funding to warrant even short-term simulations at a specific site. Therefore, there is a need for developing a means for economically applying general information from a series of short-term numerical simulations of sediment transport at generic sites under a variety of conditions occurring at specific sites.

318. This development can be accomplished by separating the work into several tasks. Definition of the range of site characteristics likely to be encountered in a field site should be the first task. These characteristics should be defined for both open-ocean and estuarine environments. The frequency of occurrence of typical energy levels must be estimated and the range of geometric settings addressed. These parameters include: tidal elevations, tidal currents, storm surge, storm currents, wind magnitude and direction, wave conditions, bottom sediment characteristics, and water depth.

319. From the results of the first task, a series of generic disposal sites should be developed for sediment transport modeling. These generic sites should be modeled with a variety of energy sources appropriate for either ocean or estuarine environments to provide input for nomograph

development. In the second task, a program of generic simulations should be designed by identifying significant parameters, nondimensionalizing these parameters, and conducting test simulations to define the range of sensitivity of the results to variation of the parameters. As the generic simulations are completed, development of the nomographs will begin. The development of nomographs from the results of the simulations, the computerization of the nomograph relationships, and evaluation of the nomograph approach by application to specific field sites and further refinement of the approach would complete the third task.

320. The development of the nomographs and procedures discussed above may require several years but is probably the most cost-effective, yet technically defensible, means of evaluating erosional characteristics of DMADS. In the interim, steady-state and the time- and rate-dependent analytical methods for estimating erosion and mound size could be further developed. These methods can be adapted for use on microcomputers and eventually added to the ADDAMS system (Hayes et al., in preparation). Before implementing analytical tools, the approach should be reviewed by other sediment transport experts to achieve a consensus since these methods have not been fully developed and verified.

321. The above developments would address questions dealing with erosion, mound size, and site capacity; however, they do not address questions concerning the transport paths and redeposition (because these processes are strongly dependent on site conditions). To address these issues, the analytical plume models could be programmed for use on microcomputers. This could be done easily, and the models would be easy to use. Although these simple models do not include settling of sediment, they give estimates of plume concentration contours; deposition patterns could be inferred from this information. Perhaps a better alternative would be to modify the disposal models to simulate the spread of resuspended sediment in addition to sediment from disposal operations. The disposal models already include settling. Physical and numerical modeling and field monitoring are other means of addressing redeposition should time and funding permit.

322. Other future research that would provide better means of evaluating the capacity of disposal sites includes the following:

- a. Improved techniques for more accurate measurement of deposition and erosion rates are needed. Present survey methods are never more accurate than ± 0.5 ft, and are in

actual practice rarely more accurate than ± 1.0 ft. Often these errors are systematic, not random, so that computations of erosion and deposition from surveys can be seriously in error.

- b. Better physical descriptions are needed of the armoring process in sediment beds and of the variation of fine sediment characteristics with time and stress history. Most of the mathematical relationships describing these processes are strongly empirical and may not represent actual conditions in the field. The improved armoring descriptions would be used to improve predictions made by numerical models and the analytical techniques.
- c. Improved techniques are required for determining the critical shear stress for fine cohesive sediment.
- d. For the limited number of DMADS that are not readily amenable to generic analyses (such as nomographs and analytical tools) or for which the political/economic climate requires a more in-depth study, a site-specific numerical model study should be employed. Although numerical models exist that could be used for modeling DMADS, these models need to be adapted and exercised, improved/updated, verified, and made accessible to the field offices.
- e. Field sites must be monitored to provide data to verify the predictive tools proposed herein.

REFERENCES

- Ackers, P., and White, W. R. 1973. "Sediment Transport: New Approach and Analysis," Journal of the Hydraulics Division, American Society of Civil Engineers, No. HY11, pp 2041-2060.
- Applied Science Associates, Inc. 1983. Proceedings, Workshop for the Development of a Scientific Protocol for Ocean Dump Site Designation, US Environmental Protection Agency, Office of Water Regulations and Standards, Criteria and Standards Division, Washington, DC.
- Ariathurai, R. 1982. "Two and Three Dimensional Models for Sediment Transport," RMA 9180, Resource Management Associates, Lafayette, Calif.
- Ariathurai, R., and Arulanadan, K. 1978. "Erosion of Cohesive Soils," Journal of the Hydraulics Division, American Society of Civil Engineers, Vol 104, No. HY2, pp 279-283.
- Ariathurai, R., et al. 1977. "Mathematical Model of Estuarial Sediment Transport," Technical Report D-77-12, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Bagnold, R. A. 1946. "Motion of Waves in Shallow Water, Interaction Between Waves and Sand Bottom," Proceedings, Royal Society of London, Series A, Vol 187, pp 1-15.
- Barnard, W. D. 1978. "Prediction and Control of Dredged Material Dispersion Around Dredging and Open-Water Pipeline Disposal Operations," Technical Report DS-78-13, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Basco, D. R., et al. 1974. "Assessment of the Factors Controlling the Long-Term Fate of Dredged Material Deposited in Unconfined Subaqueous Disposal Areas," Contract Report D-74-8, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Bokuniewicz, H. J., et al. 1978 (Apr), "Field Study of the Mechanics of the Placement of Dredged Material at Open-Water Disposal Sites," Technical Report D-78-7, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Boone, C. G., et al. 1978. "Aquatic Disposal Field Investigations, Columbia River Disposal Site, Oregon, Evaluative Summary," Technical Report D-77-30, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Briggs, R. B. 1970. Geology and Hydrography; No. 3, Cross Physical and Biological Effects of Overboard Spoil Disposal in Upper Chesapeake Bay, L. E. Cronin et al., eds., Natural Resources Institute, University of Maryland, College Park, Md.
- Brogdon, N. J., Jr. 1969. "Effects of Proposed Barriers on Hurricane Surge Heights: Hydraulic Model Investigation; Report 1, Galveston Bay Hurricane Surge Study," Technical Report H-69-12, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Butler, H. L. 1978. "Numerical Simulation of the Coos Bay-South Slough Complex," Technical Report H-78-22, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

- Dingler, J. R., and Inman, D. L. 1976. "Wave-Formed Ripples in Nearshore Sands," Proceedings, Fifteenth Coastal Engineering Conference, Honolulu, Hawaii, Vol II, pp 2109-2126.
- Eagleson, P. S., et al. 1958. "The Mechanics of the Motion of Discrete Spherical Bottom Sediment Particles Due to Shoaling Waves," Technical Memorandum No. 104, US Army Engineer Beach Erosion Board, Washington, DC.
- Edinger, J. E., and Buchak, E. M. 1981. "Estuarine Laterally Averaged Numerical Dynamics: The Development and Testing of Estuarine Boundary Conditions in the LAEM Code," Miscellaneous Paper EL-81-9, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Fisackerly, G. M., et al. "The Atchafalaya River Delta; Report 2, Field Data," Technical Report HL-82-15 (Report 2, in preparation), US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- George, T. S., and Walton, R. 1984. "Equation Workbook: Scientific Protocol for Ocean Dump Site Designation," US Environmental Protection Agency, Criteria and Standards Division, Washington, DC.
- Graf, W. H. 1971. Hydraulics of Sediment Transport, McGraw-Hill Book Company, New York.
- Gularte, R. C., et al. 1977. "Threshold Erosional Velocities and Rates of Erosion for Redeposited Estuarine Dredge Materials," Proceedings, Second International Symposium on Dredging Technology, Paper H3, Texas A&M University, College Station, Tex.
- Harrison, A. S. 1950. "Report on Special Investigation of Bed Sediment Segregation in a Degrading Bed," Series 33, Institute for Engineering Research, University of California, Berkeley, Calif.
- Hayes, M. O., and Scott, A. J. 1964. "Environmental Complexes, South Texas Coast," Transactions, Gulf Coast Geological Society Association, Vol 14, pp 237-240.
- Hayes, D. F., et al. "ADDAMS: Automated Dredging and Disposal Alternatives Management System" Instruction Report (in preparation), US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Horikawa, K., and Watanabe, A. 1968. "Laboratory Study on Oscillatory Boundary Layer Flow," Proceedings, Eleventh Conference on Coastal Engineering, London, England, Vol I, pp 467-486.
- Hudson, R. Y., et al. 1979. "Coastal Hydraulic Models," Special Report No. 5, US Army Corps of Engineers, Coastal Engineering Research Center, Fort Belvoir, Va.
- Jensen, R. E. 1983. "Atlantic Coast Hindcast, Shallow-Water, Significant Wave Information," Wave Information Study (WIS), Report 9, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Johanson, E. E., et al. 1976. "State-of-the-Art Survey and Evaluation of Open-Water Dredged Material Placement Methodology," Contract Report D-76-3, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Johnson, B. H. "User's Guide for Dredged Material Disposal Models for Computing the Short Term Physical Fate at Open-Water Sites," Instruction Report (in preparation), US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Johnson, B. H., Boyd, M. B., and Keulegan, G. H. "A Mathematical Study of the Impact on Salinity Intrusion of Deepening the Lower Mississippi River Navigation Channel," Technical Report (in preparation), US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Jonsson, I. G. 1965. "Friction Factor Diagrams for Oscillatory Boundary Layers," Basic Research Progress Report No. 10, Part 2, Coastal Engineering Laboratory, Technical University of Denmark, Copenhagen.

Jonsson, I. G. 1978. "A New Approach to Oscillatory Rough Turbulent Boundary Layers," Series Paper No. 17, Technical University of Denmark, Copenhagen.

Kamphuis, J. W. 1975. "Coastal Mobile Bed Model--Does it Work?" Proceedings, Second Annual Symposium on Modeling Techniques, San Francisco, Calif. pp 993-1009.

King, I. P. 1982. "A Three Dimensional Finite Element Model for Stratified Flow," Proceedings, Fourth International Symposium on Finite Elements in Fluids, Tokyo, Japan.

Komar, P. D., and Miller, M. C. 1974. "Sediment Threshold Under Oscillatory Waves," Proceedings, Fourteenth Coastal Engineering Conference, Copenhagen, Denmark, Vol II, pp 756-775.

Krone, R. B. 1972. "A Field Study of Flocculation as a Factor in Estuarial Shoaling Processes," Technical Bulletin No. 19, US Army Corps of Engineers, Committee on Tidal Hydraulics, Washington, DC.

_____. 1978. "Aggregation of Suspended Particles in Estuaries," Estuarine Transport Processes, B. Kjerfve, ed., University of South Carolina Press, Columbia, S. C., pp 177-190.

Leendertse, J. J. 1967. "Aspects of a Computational Model for Long-Period Water Wave Propagation," RM-5294-PR, The Rand Corporation, Santa Monica, Calif.

_____. 1970. "A Water Quality Simulation Model for Well-Mixed Estuaries and Coastal Seas; Vol 1, Principles of Computation," RM-6230-RC, The Rand Corporation, Santa Monica, Calif.

Leendertse, J. J., and Liu, S. K. 1975. "A Three-Dimensional Model for Estuaries and Coastal Seas; Vol 2, Aspects of Computation," R-1764-OWRT, The Rand Corporation, Santa Monica, Calif.

Lenhoff, L. 1982. "Incipient Motion of Particles Under Oscillatory Flow," Proceedings, Eighteenth Coastal Engineering Conference, Cape Town, Republic of South Africa, Vol II, pp 1555-1568.

Letter, J. V., Jr., and McAnally, W. H., Jr. 1977. "Physical Hydraulic Models: Assessment of Predictive Capabilities; Report 2, Movable-Bed Model of Galveston Harbor Entrance," Research Report H-75-3, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Letter, J. V., Jr., and McAnally, W. H., Jr. 1981. "Physical Hydraulic Models: Assessment of Predictive Capabilities; Report 3, Model Study of Shoaling, Brunswick Harbor, Georgia," Research Report H-75-3, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Madsen, O. S., and Grant, W. D. 1975, "The Threshold of Sediment Movement Under Oscillatory Waves: A Discussion," Journal of Sedimentary Petrology, Vol 45, pp 360-361.

Manohar, M. 1955. "Mechanics of Bottom Sediment Movement Due to Wave Action," Technical Memorandum No. 75, US Army Engineer Beach Erosion Board, Washington, DC.

May, E. B. 1973. "Environmental Effects of Hydraulic Dredging in Estuaries," Bulletin No. 9, Alabama Marine Resources, Dauphin Island, Ala.

McAnally, W. H., et al. 1984a. "Columbia River Hybrid Modeling System," Journal of Hydraulic Engineering, American Society of Civil Engineers, Vol 110, No. 3, pp 300-311.

_____. 1984b. Application of Columbia Hybrid Modeling System," Journal of Hydraulic Engineering, American Society of Civil Engineers, Vol 110, No. 5, pp 627-642.

Mehta, A. J., and Partheniades, E. 1982. "Resuspension of Deposited Cohesive Sediment Beds," Proceedings, Eighteenth Coastal Engineering Conference, Vol II, Cape Town, Republic of South Africa, pp 1569-1588.

Migniot, C., et al. 1975. "LCHF Coastal Sediment Modeling Techniques," Proceedings, Second Annual Symposium on Modeling Techniques, San Francisco, Calif., pp 1638-1657.

Moherek, A. J. 1978. "Flume Experiments on Sand, Silt, and Clay Mixtures from the Offshore Dredged Material Disposal Site, Galveston, Texas," Technical Report D-78-34, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Morton, R. W., and Robertson, R. 1983. "A Disposal Site Classification Technique," Progress Report 1983, Contribution No. 34, Science Applications, Inc., Ocean Science and Technology Division, Newport, R. I.

Nielsen, P. 1979. "Some Basic Concepts of Wave Sediment Transport," Series Paper No. 20, Technical University of Denmark, Lyngby.

Noda, E. K. 1972. "Equilibrium Beach Profile Scale-Model Relationship," Journal of the Waterways, Harbors, and Coastal Engineering Division, American Society of Civil Engineers, Vol 98, No. WW4, pp 511-528.

Norton, W. R., and King, I. P. 1977. "Operating Instructions for Computer Program RMA-2: A Two-Dimensional Finite Element Program for Problems in Horizontal Free Surface Hydrodynamics," Resource Management Associates, Lafayette, Calif.

Norton, W. R., et al. 1973. "A Finite Element Model for Lower Granite Reservoir," Water Resources Engineers, Walnut Creek, Calif.

Parker, W. R., and Kirby, R. 1981. "Time Dependent Properties of Cohesive Sediment Relevant to Sedimentation Management: European Experience," Estuarine Comparisons, V. S. Kennedy, ed., Academic Press, New York.

- Parthenaides, E. 1962. "A Study of Erosion and Deposition of Cohesive Soils in Salt Water," Ph.D. Dissertation, University of California, Berkeley, Calif.
- Pequegnat, W. E. 1978. "An Assessment of the Potential Impact of Dredged Material Disposal in the Open Ocean," Technical Report D-78-2, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Poindexter, M. E. "Behavior of Subaqueous Sediment Mounds: Effect on Dredged Material Disposal Site Capacity," Technical Report (in preparation), US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Postma, H. 1967. "Sediment Transport and Sedimentation in the Estuarine Environment," Estuaries, C. H. Lauff, ed., No. 83, American Association for the Advancement of Science, pp 158-179.
- Ragsdale, D. S. 1983. "Sea-State Engineering Analysis System (SEAS)," Wave Information Study (WIS), Report 10, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Rance, P. J., and Warren, N. F. 1968. "The Threshold of Movement of Coarse Material in Oscillatory Flow," Proceedings, Eleventh Conference on Coastal Engineering, London, England, Vol I, pp 487-491.
- Riedel, H. P., et al. 1972. "Measurement of Bed Shear Stress Under Waves," Proceedings, Thirteenth Coastal Engineering Conference, Vancouver, British Columbia, Canada, Vol I, pp 587-603.
- Sager, R. A., and Hales, L. Z. 1979. "Inlets, Part VII, Coastal Hydraulic Models," Special Report No. 5, US Army Engineer Coastal Engineering Research Center, Fort Belvoir, Va.
- Saila, S. B., et al. 1972. "Results of Studies Related to Dredged Sediment in Rhode Island Sound," Proceedings, Annual North-Eastern Regional Anti-Pollution Conference, Newport, R. I., pp 71-80.
- Salem, M. A., and Krizek, R. J. 1973. "Consolidation Characteristics of Dredging Slurries," Journal of the Waterways, Harbors, and Coastal Engineering Division, American Society of Civil Engineers, Vol 99, No. WW4, pp 439-458.
- Schlichting, H. 1968. Boundary-Layer Theory, McGraw-Hill, New York.
- Schmalz, R. A., Jr. 1985. "Numerical Model Investigation of Mississippi Sound and Adjacent Areas," Miscellaneous Paper CERC-85-2, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Sheng, Y. P. 1983. "Mathematical Modeling of Three-Dimensional Coastal Currents and Sediment Dispersion: Model Development and Application," Technical Report CERC-83-2, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Sheng, Y. P. 1984. "Preliminary User's Manual, 3-D Mathematical Model of Coastal, Estuarine, and Lake Currents (CELC3D)," Instruction Report D-84-1, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.
- Sheng, Y. P., and Butler, H. L. 1982. "Modeling Coastal Currents and Sediment Transport," Proceedings, Eighteenth Coastal Engineering Conference, Cape Town, Republic of South Africa, Vol II, pp 1127-1148.
- Shields, A. 1936. "Anwendung der Ahnlichkeits mechanik und der Turbulenzforschung auf die Geschiebebewegung," No. 26, Mitteilungen der Preussischen Versuchsanstalt fur Wasserbau und Schiffbau, Berlin, Germany.

Simmons, H. B. 1966. "Field Experience in Estuaries," Estuary and Coastaline Hydrodynamics, Chap 16, A. T. Ippen, ed., McGraw-Hill, New York, pp 673-690.

Simmons, H. B., and Boland, R. A., Jr. 1969. "Model Study of Galveston Harbor Entrance, Texas," Technical Report H-69-2, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Simmons, H. B., et al. 1971. "Predicting Construction Effects by Tidal Modeling," Miscellaneous Paper H-71-6, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Sleath, J. F. A. 1978. "Measurement of Bed-Load in Oscillatory Flow," Journal of the Waterways, Port, Coastal, and Ocean Division, American Society of Civil Engineers, Vol 104, No. WW4, pp 291-307.

Sternberg, R. W. 1972. "Predicting Initial Motion and Bedload Transport of Sediment Particles in the Shallow Marine Environment," Shelf Sediment Transport: Processes and Patterns, Dowden, Hutchinson, and Ross, Inc., Stroudsburg, Pa.

Sternberg, R. W., et al. 1977. "Aquatic Disposal Field Investigation, Columbia River Disposal Site, Oregon; Appendix A: Investigation of the Hydraulic Regime and Physical Nature of Bottom Sedimentation," Technical Report D-77-30, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Swart, D. H. 1974. "Offshore Sediment Transport and Equilibrium Beach Profiles," Publication No. 131, Delft Hydraulics Laboratory, Delft, The Netherlands.

_____. 1976. "Coastal Sediment Transport: Computation of Longshore Transport," Report No. R968, Part 1, Delft Hydraulics Laboratory, Delft, The Netherlands.

Swart, D. H., and Fleming, C. A. 1980. "Longshore Water and Sediment Movement," CSIR Technical Report SEA/8009, National Research Institute for Oceanography, Stellenbosch, Republic of South Africa.

Tetra Tech, Inc. 1984. "Report on the Study of Currents at the Alcatraz Disposal Site, San Francisco Bay-Delta Model," Pasadena, Calif.

Thomas, W. A., and McAnally, W. H., Jr. 1985. "User's Manual for the Generalized Computer Program System: Open-Channel Flow and Sedimentation; TABS-2, Main Text," Instruction Report HL-85-1, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Trawle, M. J., and Johnson, B. H. 1986. "Alcatraz Disposal Site Investigation," Miscellaneous Paper HL-86-1, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

US Army Corps of Engineers. 1969. "Guidelines for Evaluating Estuary Studies, Models, and Comprehensive Planning Alternatives," Washington, DC.

_____. 1983. "Dredging and Dredge Material Disposal," Engineer Manual EM 1110-2-5025, Washington, DC.

US Army Corps of Engineers, Coastal Engineering Research Center. 1984. Shore Protection Manual, 4th ed., Vols I and II, US Government Printing Office, Washington, DC.

US Army Engineer Division, New England. 1984. "Disposal Area Monitoring System (DAMOS), Summary of Program Results; Volume I: Overview of DAMOS Program," Boston, Mass.

US Army Engineer Hydrologic Engineering Center. 1977. "Scour and Deposition in Rivers and Reservoirs; User's Manual," Report HEC-6, Davis, Calif.

Whalin, R. W., and Chatham, C. E., Jr. 1979. Coastal Erosion, Part V; Coastal Hydraulic Models," Special Report No. 5, US Army Engineer Coastal Engineering Research Center, Fort Belvoir, Va.

APPENDIX A: FIELD AND LABORATORY TESTS

1. Field and laboratory investigations are fundamental and essential aspects of evaluating the long-term behavior of dredged material placed in any aquatic environment (riverine, estuarine, lacustrine, coastal, and oceanic). The degree, time, complexity, and associated costs for conducting such studies can vary through several orders of magnitude. The attributes of interest are similar to those of any other study involving the transport of naturally placed sediment in addition to attributes specifically related to or initiated by the disposal operation, i.e., mudflows, turbidity plumes, consolidation, and grain-size equilibrium. In general, field and laboratory studies are also used to complement or verify other methods of study (analytical or modeling) available for predicting the long-term fate of deposited dredged material and to determine the useful life and capacity of an aquatic disposal site.

2. A thorough review of sediment transport processes and physical attributes of interest is recommended but is beyond the scope of this appendix. The reader is referred to any basic text on the fundamentals of fluid mechanics and transport processes. Graf's (1971)* book on the hydraulics of sediment transport is a good starting point since it also provides a section on sediment measuring devices. Although each of the aquatic environments that receives disposed dredged material is unique, field and laboratory procedures followed in investigating sediment transport are quite similar. The methods and procedures discussed in this appendix are generally applicable to each environment. The type of material disposed is of utmost importance in designing and implementing meaningful field and laboratory studies. Different processes, procedures, and attributes apply to cohesive and noncohesive sediment types.

3. In earlier sections, methods for evaluating the long-term behavior of aquatically disposed dredged material were described. Various characterizations and field observations were suggested as part of those methods. This appendix presents two general approaches for the direct study of the long-term transport and fate of aquatically disposed dredged material: field and laboratory tests. Techniques for designing such tests for a specific project are outlined.

* See References at the end of the main text.

4. Field and laboratory tests form a class of methods that range from full-scale field to small-scale laboratory experiments. They provide a description of some phase of the long-term fate of dredged material. In this sense, they are independent of the analytical and modeling methods described earlier. However, good practice would use those other methods to extend field and laboratory tests to conditions not directly observed. Also, a principal reason for performing field and laboratory tests is to verify information derived from other methods.

5. The limitations of field and laboratory methods are less theoretical and more practical than for the analytical and modeling methods outlined in earlier sections. Field tests are expensive to conduct; therefore, the conditions under which the observations are made should be carefully chosen and thoroughly reported. Although characterization tests are not the subject here, the section "Laboratory Tests" (paragraphs A34-A49) provides some suggestions for characterizing sediments that may be useful for extending the results obtained from laboratory tests.

Field Tests

6. Field tests encompass any prototype-scale experiments made to determine the behavior of disposed dredged material. Tests can include disposal operations for an ongoing project or scaled-down, carefully controlled disposal. The tests can include studies of the movements of natural or tracer materials. Since the long-term transport and fate of material are dependent upon the short-term behavior (i.e., the way the material is convected and settles to the bed), all time and space scales are appropriate to these studies.

Objectives

7. Near-field experiments are conducted to define the extent and nature of the plume during its descent; the surge and collapse after the material impacts the bed; the turbidity that escapes from the disposal site; and the areal distribution, thickness, and makeup of material on the bed.

8. Long-term experiments are conducted to define the path of turbidity that escapes during the disposal operation and the ultimate fate of sediment associated with the turbidity. Experiments are also conducted to determine resuspension and transport of deposited material from the disposal site and

its ultimate fate. Long-term experiments are conducted to record the stability of the disposal mound and vertical processes such as consolidation or mixing that take place there.

Measurements

9. Concentration. The standard method for determining suspended sediment concentration is by measuring nonfilterable solids by gravimetric means. Samples can be collected in the field using pump and hose arrangements or with various sampling devices. The preferred method of drawing samples is by the isokinetic method. That is, the sample is removed from the environment at the same velocity as the ambient. This can be performed using a nozzle pointing into the flow and matching the nozzle intake velocity with that of the ambient. A water sampler can be used if its flow-through alignment can be adjusted to that of the ambient flow.

10. For samples of suspensions with a high concentration of bed sediment, the standard analysis is total solids or bulk wet density. When the material to be sampled is not moving, samples can be captured or core samples collected and subsampled. If the material is in motion, the same guidance given above applies.

11. In-situ methods are often used to determine concentration. This allows greater density of sampling and usually provides a real-time indication of concentration. This can be an advantage because it allows adjustments to the sampling design in response to the real-time data. Methods include:

- a. Nuclear. Gamma ray attenuation or scatter has been used to determine concentration. It has the advantage of not requiring special calibration.
- b. E-magnetic. X-ray attenuation has been used. Time-domain reflectometry has recently been developed to measure water content in porous soil.
- c. Optical. Turbidity, beam attenuation, and nephelometry have been used to trace and determine the concentrations of suspensions. They require a special calibration for each sediment encountered since the scattering properties measured depend on the particle size and are affected by the presence of dissolved (organic) materials. Near-forward scattering has also been measured and can be related to concentration without some of these limitations.
- d. Fluorometry. Certain natural or introduced materials fluoresce when excited at specific light bands. Their concentrations can be related to optically determined fluorescence.

12. Settling velocity. Settling velocity is perhaps the most important parameter governing the transport and deposition of sediment. It is also discussed in the section "Laboratory Tests" (paragraphs A34-A49). Since settling velocities are a function of the state of the suspension, measurements of settling velocity made in the field have been found to be most representative of in situ values. Field experiments to determine settling are performed with settling tubes or columns or with partitioned columns. If the material is coarse grained, its settling velocity characteristics can be deduced from grain-size data.

13. Mass and density. Obtaining accurate records of the total mass of sediment discharged is a crucial first step in determining the fate of dredged material placed in an aquatic environment. The initial bed density (vertical structure) and extent of a freshly deposited disposal mound are required to determine the amount of material carried away by ambient currents during the disposal process. The variation in bed density over time is another important parameter that should be monitored. This is especially critical when monitoring the consolidation of cohesive material because of its influence on erosional processes. Unfortunately, instrumentation is unavailable to easily record and monitor consolidation, although settlement plates have been used to infer mass and density changes.

14. Positioning control. High-accuracy horizontal control of position is required during bathymetric surveys, for the location of marker buoys, for the location of instrument arrays, and for measurement stations in the near field. Microwave transponder systems are generally used for this purpose. Other systems with similar accuracy (± 3 m) can be used. Arrays of submerged acoustic transponders may be used in areas where the location of shore stations is not possible.

15. Depths. If depth soundings are to be used for determining the thicknesses and stability of deposited material at a disposal site, high-quality survey techniques must be used. The vertical datum from which the measurements are referenced (i.e., the local water surface) must be monitored at the station(s) as close to the site as possible. The datum should be based on observed water heights, not predicted water heights. If sufficient wave action or ocean swell is present at the site, automatic corrections for heave may be required. With sufficient care, bathymetric survey accuracy can be 0.2 ft; however, achieving such accuracy using standard depth sounders is very

difficult and is not usually achieved in open-water environments. More realistically, achievable accuracy in open water is 0.5 ft. As an additional control, survey lines should be extended into areas where the bottom is undisturbed.

16. Thicknesses. Fathometers, especially low-frequency units, can record echoes from subsurface horizons. However, this technique is not useful in determining deposit thickness unless the thickness is relatively large. Side-scan sonar can be used to measure bed relief and net transport directions.

17. Direct measurements of the thickness of deposited material can be made if bathymetric surveys are infeasible. Gravity or vibracoring devices can be used. Artificial horizons can be created by spreading the surface with tracer material if natural horizons are indistinct. Stakes or other markers can be installed and later read by divers.

18. Velocity. Current speed and direction are measured in connection with the study of surges and ambient currents. A wide variety of measurement devices are commercially available, many of which are suitable for this purpose.

19. For the measurement of surges resulting from dredged material disposal, current sensors are usually mounted to the bottom or on tight-wire moorings. It may be necessary to have several sensors in the vertical if the extent, as well as characteristic velocities, is of interest. Sensors so placed should be exceptionally robust, and their calibrations should not be unduly affected by relatively high concentrations of solids.

20. Time-series measurements of ambient currents are usually made from tight-wire moorings. Multiple sensors are usually needed to adequately resolve the vertical structure of mass transport. If waves are present, sensors should not rectify them into a spurious flow. Water levels should also be measured at current meter mooring sites so that accurate mass-transport vectors can be computed.

21. Ambient currents can be measured from survey boats. It is usually necessary to moor boats to prevent drift error. Several anchors may be required. If drift can be accurately measured, correction can be made to drifting boat observations. The best arrangement is to use a string of sensors to simultaneously measure currents at multiple depths, especially in

unsteady flow. Alternately, a single sensor can be used to profile vertically.

22. Acoustic instruments have been recently developed to profile currents (or apparent currents if the boat is drifting) by beam reflection. The signal is gated in vertical sections on the order of 10 ft or less by these instruments. Acoustic current meters that measure currents over a range of depths operate by measuring Doppler frequency shifts.

23. Shore-mounted microwave systems have been recently developed that are claimed to be capable of measuring surface currents over a wide area and at high sampling rates.

24. Large-scale current structure can be inferred from satellite imagery for many areas. These patterns can be captured at time scales of 12 hr or less and over longer time periods. Atmospheric conditions can interrupt coverage. The depth to which currents are resolved depends on local turbidity.

25. Drogues can be used effectively to measure currents in the Lagrangian frame of reference. Lagrangian currents are of greater interest in mass-transport studies than are the Eulerian currents measured at a point. They can be determined from a system of moored instruments and calculated spatial derivatives or by following the path of a water particle using some types of drogue.

26. Drogues can be launched in groups at preset depths and followed by chase boat or by radar. The rate of spread of drogues set to the same depth can be used to estimate horizontal diffusion rates. Expendable drogues with return labels are used to study surface and bottom residual flows and transport. They depend on public cooperation and an appropriate physical setting for success.

27. Sediment distribution. The sediment grain-size distribution or some other distinct characteristic (mineralogy, color, shape) can be used to interpret the hydrodynamic characteristics of the disposal site and/or to monitor the transport of disposed material from the disposal site. Measurements of sediment grain-size distributions are not routinely performed during field tests. Samples of bed (bottom) material are collected and returned to soil testing laboratories for detailed analysis, including other pertinent soil mechanics parameters such as bulk density, liquid limit, consolidation, and shear strength. Details of such analyses are presented in "Laboratory Tests" (paragraphs A34-A49).

28. Bed load. The bed-load transport rate of coarse-grained material can be measured with bed-load traps or lined pits or by instruments that record grain-impaction frequency. Results from past bed-load traps are somewhat suspect. The Coastal Engineering Research Center, located at the US Army Engineer Waterways Experiment Station, is presently developing an improved trap.

29. Visual observations. Photographic or other visual records can be useful in describing the behavior of disposed material at the time of disposal or later. Remote or diver-held cameras or underwater television units can be used. Fine-scale observations of current and wave ripples indicating strength and direction can be obtained. Section photographs of near-surface sediment have been made with special coring devices by REMOTS (Remote Ecological Monitoring of the Seafloor) as part of DAMOS (Disposal Area Monitoring System) developed by US Army Engineer Division, New England (1984). The REMOTS camera system provides information combining the distribution of dredged material, measurement of the physical properties of the sediment, limited chemical data, and substantial insight into the benthic population. Interpretation of REMOTS photographs has added significantly to the overall understanding of disposal sites in Long Island Sound.

Design

30. Available background data for the site and its vicinity, and studies on similar disposal operations, should be reviewed before a field study is designed. The objectives of the study should be clearly identified and a reasonable experimental design developed.

31. The behavior under study probably will depend on a number of environmental (physical) factors. These factors should all be considered in the experimental design, even though they may have little or nothing to do with the disposal operation. Arrangements should be made to obtain as much monitoring data as possible on the environmental variables.

32. A data quality control program should be considered as part of the experimental design. This should include calibration and operational verification on all instruments used. Duplicate, spiked, and standard samples should be used as checks on analytical and sampling methods. Custody logs should be maintained for each sample.

33. The study design should be coordinated with other work at the disposal site or in the area. Costs can be reduced if data can be shared. Sample sharing may be possible if sample handling is appropriate to both studies.

Laboratory Tests

Objective

34. Laboratory tests are carried out to observe some behavior of the sediment disposed at a site or of the sediment existing at or near the site. It is necessary to collect representative samples of these sediments and of the native water into which they are discharged. Laboratory tests classify or characterize, provide model input data, and check model behavior.

Measurements

35. Characterization. The sediment involved in field or laboratory tests should be well characterized, especially fine-grained material. Characterization will allow the results of these tests to be extended to other situations and can improve the interpretation of the results.

36. Appropriate parameters for characterization should relate to other important behaviors of sediment, such as shear strength and settling velocities. These parameters should be determined for the sediment, suspension water, pore water, and sediment bed structure. Standard methods of analysis should be used:

a. Sediment.

- (1) Sediment grain-size distribution.
- (2) Organic content.
- (3) Cation exchange capacity.
- (4) Oil and grease content.

b. Suspension and pore waters.

- (1) Salinity or total filterable solids.
- (2) Sodium absorption ratio.
- (3) pH.

c. Sediment beds.

- (1) Bulk density variation with depth.
- (2) Liquid limit and percent moisture.

37. Settling tests. For coarse sediment, settling velocity can be inferred from the grain size with fairly good confidence. Fine-grained sediment, however, has settling characteristics that depend on physicochemical properties other than particle size. Settling of fine-grained sediment is also related to the concentration of the suspension and to properties of the

flow (i.e., salinity, temperature, turbulence, and flow depth). Settling velocities can vary by orders of magnitudes for particles in a single sample. In those cases, a mean settling velocity is insufficient to characterize the settling of the sample. It is best to determine the distribution of settling velocities or at least the variance of the distribution.

38. Settling velocity of fine-grained sediment varies with concentration in three ranges. At low concentrations, less than about 1 to 200 mg/l, settling velocities are insensitive to concentration (free range). At concentrations above 200 mg/l and below 1 to 5 g/l, settling velocities are enhanced with concentration. Above the 5-g/l concentration, settling velocities are hindered or reduced with increases in concentration. Settling tests should cover the range of concentrations that exist in the environment, and should be performed with native water.

39. Settling tests for the free and enhanced ranges are usually performed by the accumulation or pipette methods, using settling tubes or columns. Fine-grain settling has been observed to vary with settling height up to about 2 m. Settling columns are therefore usually about 2 m high. Sediments are mixed to the desired concentration and agitated for a standard length of time. The mixture is introduced into the settling tube, and sampling is begun. Bottom withdrawal and sampling at a fixed distance below the surface are used. Data are then analyzed according to the method used.

40. Settling tests in the hindered-settling range are sometimes called flocculent or zone settling tests because of the unusual behavior of the sediment suspension. During these tests, a distinct interface forms with the sediment below and clear water above. Tests are performed in columns and, usually, the only data recorded are the levels of the interface with time, usually 25 to 100 hr. The column should be about 8 in. in diameter if concentrations are above about 50 g/l to avoid sidewall effects. It does not have to be deep, as results are insensitive to depths over about 1 ft. A range of concentrations should be run to examine the effect on settling rate.

41. Erosion and deposition tests. These tests are carried out in flows to determine the lowest (highest) bed shear stresses at which erosion (deposition) occurs. The rate of erosion (deposition) is also determined.

42. Erosion tests on dredged material should be performed on so-called deposited beds, as this state of consolidation is representative of material that has been dredged, mixed with water, and settled. Erosion tests on

consolidated or partially consolidated sediment would use remolded or cored sediment that has a relatively low moisture content.

43. Studies on erosion of newly deposited material and on deposition require a recirculating flume. A suspension is created and the flow is reduced, usually in steps, to allow a bed to form. Visual observations of the bed should be made, especially if mixtures of cohesive and noncohesive materials are being studied. The flume should have minimal secondary currents, and pumps should not destroy the flocculent structure of the sediment. Usually, concentration in the flow is measured with time, along with the bed shear stress (corrected for sidewall effects). Thicknesses or densities of deposited material are also measured.

44. Apparently, except for a study by Moherrek (1978) for the DMADS at Galveston, Tex., no reports have been made of studies dealing directly with sediment mixtures containing near-equivalent amounts of sand, silt, and clay which have been obtained directly from DMADS. Two-dimensional, rectangular, circulating flume experiments were performed using four sand/silt/clay mixtures sampled from the offshore Galveston site in order to determine the critical erosion velocity, shear stress, and modes of sediment transport for each mixture. Also, an analysis of the hydrographic regime for offshore Galveston was performed based on meteorologic and oceanographic data collected between February 1975 and June 1976. Results of the flume experiments and hydrographic analysis were extrapolated to sediment transport processes believed operative in the offshore disposal site.

45. Application of experimentally determined critical erosion and deposition velocities as measures of the critical velocities occurring within an offshore disposal site is valid under certain assumptions. These assumptions include: (a) bottom currents in the offshore dredged material disposal site are characterized by uniform, fully developed turbulent flow; (b) the effect of wave-induced oscillatory motion on the threshold of cohesive sediment movement is negligibly small; and (c) the hydrodynamic roughness exhibited by the dredged material placed into the flume is similar to the roughness on the offshore disposal mounds. Establishment of the validity and/or variability of these three assumptions requires the development of new field-monitoring techniques to measure the in situ bottom current and sediment conditions over long time periods. Until such techniques are developed,

extrapolation of the critical velocity results to field conditions should be viewed as a best estimate based upon state-of-the-art methods.

46. It should be noted that the experimental flume tests with the samples of dredged material from the Galveston DMADS were conducted with unidirectional flow only, without benefit of wave or storm effects. A discussion of climatic effects upon sediment transport processes for offshore Galveston would be incomplete without consideration of severe tropical storms. Although no hurricanes seriously affected the Galveston area during the period of the study by Moherrek (1978), destructive storms such as Hurricane Audrey (1957), Carla (1961), and more recent storms have produced extreme storm tides. Hurricane Carla caused a 2.3-m tidal height rise. Hayes and Scott (1964) documented the effects of the storm surge and associated currents generated by Hurricane Carla along the central Texas coast. After passage of the storm inland, seaward-flowing density currents deposited a fresh sand layer (up to 3 cm thick) over a sandy mud bottom in depths of 15 to 20 m. In addition, a graded bed (up to 10 cm thick) composed of very fine sand, silt, and clay was traced over the homogeneous mud bottom to depths exceeding 60 m. Such sedimentologic features point out the major role that a tropical storm can have in modifying an offshore geologic environment over a very short time period.

47. Threshold erosional velocities and rates of erosion were determined by Gularte et al. (1977) for material dredged from the Taunton River, Massachusetts, and the Thames River, Connecticut, prior to being placed in an open-ocean disposal site. Both of the materials tested can be classified as marbled black-olive drab, fairly well-graded silty clay of high organic content. The Taunton River material contained approximately 20 percent (by weight) clay-size particles (less than 0.002 mm) and 70 percent silt-size particles (0.06 to 0.002 mm), with the remainder fine sand and shell fragments. The material obtained from the Thames River contained much less clay-size particles (10 percent), slightly less silt-size particles (60 percent), and 30 percent sand-size material and shell fragments. The dredged material was tested in a research water tunnel designed and fabricated specifically for this research.

48. Tank tests. Studies of the entrainment rates of descending plumes or jets, the mounding and spreading of material as it impacts the bed, and the limits of bottom spreading can be studied in scale models (tank tests). The sophistication of the tests and of the tank facility varies greatly depending

on the objectives of the tests. The ambient receiving waters in the tank can be quiescent or flowing, homogeneous or stratified. The disposal can be moving or stationary, and can have varying degrees of realism in the portrayal of the vessel.

49. Tank tests should be considered if other methods of analysis are lacking or if they are untested. Scaling relationships must be carefully defined. The initial dynamic behavior has been modeled using Froude scaling.

APPENDIX B: PHYSICAL MODELING

Use of Physical Models

1. Physical models, also called hydraulic models, are scaled representations of a surface water body. The behavior of flows in the scale model is related to behavior of flows in the prototype (the natural water body that the model represents) by means of scaling relationship. Physical modeling theory and practice are thoroughly described in "Coastal Hydraulic Models" (Hudson et al. 1979*); this appendix is quoted or paraphrased from Part III, "Estuaries" of that reference.

2. All hydraulic models have one common characteristic: a model cannot be a completely accurate simulation of all of the complex phenomena present in the prototype. To approximate model-prototype similarity, a hydraulic model should reproduce the geometry and boundary roughness of the prototype and be able to simulate the following variables (individually and collectively) as they vary with time at all points in the system:

- a. Water surface elevations.
- b. Current velocities and directions.
- c. Salinities.
- d. Physical characteristics of sediment.
- e. Transportation, deposition, and scour of sediment.
- f. Parameters reflecting water quality, such as dissolved oxygen, temperature, viscosity, and diffusion of introduced pollutants.
- g. Freshwater and saltwater discharges into the system and the turbulent intermixing within the water mass.
- h. Effects of winds on setup, waves, local water currents, mixing, diffusion, etc.

3. Simulation of all of these variables is unnecessary to solve every problem. In physical model studies of certain problems, some of the variables would be completely irrelevant and others would be so nearly irrelevant as to be negligible. The recommended approach to the design of a physical model is to first select the prototype variables that would significantly affect, or be affected by, the problem to be studied (or by possible solutions), and then to design the model to simulate the selected variables with acceptable accuracy.

* See References at the end of the main text.

4. Proper scaling of tidal, Great Lakes, and coastal processes usually dictates that the model have different length scales for horizontal and vertical dimensions. Such a model is called a distorted-scale model. Modeling relationships for short-period waves usually dictate that the vertical and horizontal length scales be equal--an undistorted model.

5. Although scale relations can be determined for various variables by analytical means, there is no assurance that an unverified distorted-scale model will accurately reproduce prototype-flow conditions when comparing model and prototype observations. This is because the contribution of model roughness to energy dissipation and mixing processes is distorted with the length scales. Therefore, it is necessary to carefully adjust the model roughness until measured prototype tides, velocities, dispersion, and salinities are accurately reproduced. This process is referred to as model verification.

6. After a physical model has been properly adjusted and verified, many of the effects of a planned activity can be predicted quantitatively. The types of activities in estuaries and coastal regions normally accomplished by the US Army Corps of Engineers include:

- a. Construction of new channels or the deepening of existing channels for navigation purposes.
- b. Construction of dikes, jetties, sediment traps, and sand-bypassing facilities to alleviate sedimentation.
- c. Dredging of new tidal inlets or the stabilization and improvement of existing inlets for small-craft or deep-draft navigation.
- d. Disposal of dredged material.
- e. Construction and operation of barriers for control of flooding by storm surges (Brogdon 1969).

Some aspects of each of these activities can be studied with a physical model; however, there are limitations to the model capabilities. Additionally, some processes, such as physicochemical processes, cannot be adequately modeled with a physical model.

7. The various phenomena that can be reproduced or simulated in hydraulic models include tides, tidal currents, density currents, littoral currents, currents generated by riverflows, salinity, mass dispersion, heat dispersion, shoaling, hurricane surges, tsunami surges, and the general effects of wave and ship action on the resuspension of sediments. Although wind-induced currents and water level setup can be reproduced, it is generally

not economically feasible to construct a large estuary model in a wind tunnel with the capability of generating winds from various directions. Coriolis forces can be simulated through model rotation, but comprehensive estuary models cannot be rotated unless they are constructed to very small scales.

8. The rise and fall of a tide or surge and its progression upstream can be accurately modeled. Not only can the magnitude, phasing, and direction of currents be reproduced at a particular point, but the longitudinal, lateral, vertical, and temporal velocity distributions can be reproduced as well. The same is true for salinity. Thus, the physical model provides a time-varying, three-dimensional representation of the hydraulic and salinity regimens of the estuary.

9. Although it is possible to model the dispersion of pollutants in three dimensions and with time, the model tracer is usually a conservative dye (i.e., no decay with time). The physical model results must, therefore, be treated analytically before they can be applied to field conditions. For dispersion of thermal and other pollutant discharges, far-field dispersion can be modeled with proper model adjustments, although it is not practical to control model surface heat exchange and, thus, heat dissipation. The near-field dispersion associated with the discharge of a jet cannot be accurately reproduced in a distorted-scale model.

10. Because sediment transport processes are very complex and poorly understood, reliable sedimentation similitude relations cannot be developed. Model simulation of sediment transport is therefore empirical and depends on a trial-and-error procedure to develop an appropriate testing technique by which to reproduce known sedimentation patterns. A qualitative reproduction of shoaling patterns and distributions can be achieved by the empirical development of a movable-bed model operating technique. However, the technique does not reproduce the changes in cohesion of deposited material, the process of aggregation or dispersion of suspended sediment, suspended-sediment concentrations, flocculation, or resuspension of sediment by wave action.

11. The accepted practice of many hydraulic laboratories experienced in the art of movable-bed modeling has been discussed by Sager and Hales (1979). The practice is to construct the model to a manageable size based on space limitations and instrumentation ability and to use a readily available material for construction (usually sand), which constitutes a model scale distortion.

12. Next, the empirical process of verifying the model to reproduce prototype bed forms such as scour and deposition leads to the distortion of a second parameter. This is usually accomplished in the model by altering the wave climate, increasing or decreasing tidal flow, or changing the time scale from that given by the hydrodynamic scaling relations to an empirically selected time scale that reproduces the sedimentology (referred to as the sedimentological time scale). These are empirical solutions based on the application of scale modeling and the experience of the researcher; however, the mechanisms of most sedimentation phenomena are still not entirely understood. Several investigators have attempted to derive formal scaling laws, resulting in many modeling formulas from which to choose. According to Kamphuis (1975), because of the variety and magnitude of scale effects, modeling movable-bed material continues to be an art rather than a science.

Advantages and Disadvantages of Physical Models

13. Physical models are an important analytical tool for a number of reasons. Foremost, a number of alternatives can be evaluated much faster and much cheaper with a model than in the prototype. Prototype testing of alternatives would probably be a prohibitively expensive, major undertaking. To test a sufficient number of prototype alternatives to ensure that the final solution is the most desirable would usually be impossible (Simmons et al. 1971). In addition, model testing of various alternatives may prevent irreversible damage to the prototype system that might be caused by an unsatisfactory design.

14. The hydraulic model study method has certain other advantages. It is a highly useful method of visually demonstrating alternative plans of improvement to the public and to representatives of local, State, and Federal agencies. The model has great value in decisionmaking on improvements, providing the necessary understandable information by observation. The model can also be a research tool, and undefined problems or principles in the prototype can be discovered and solved by operation of a hydraulic model (US Army Corps of Engineers 1969).

15. The hydraulic model has shortcomings, not the least of which is the first cost for construction and verification. Changes in conditions and alternative plans are more time-consuming to study in a physical model than in

a mathematical model. The technique provides little information on suspended-sediment concentrations or on patterns of resuspension of fine-grained sediments (US Army Corps of Engineers 1969).

Field Data Requirements

16. Because of various scale effects in estuary models and the attendant need for adjusting the model roughness, a large amount of field data must be obtained to ensure that the model is capable of reproducing prototype phenomena to an acceptable degree of accuracy. This requirement is even more stringent for those phenomena that are simulated rather than modeled directly. For example, rather than reproducing a scaled wind field in the models, the effects of wind action on the mixing of fresh water and saltwater are simulated by fans that blow down on the water surface in a random pattern or by bubbling air through the water column. Similarly, sedimentation is simulated by developing an operating technique by trial-and-error that will duplicate known shoaling patterns; however, no attempt is made to actually scale the sediment or to determine the sedimentation time scale by analytical means.

17. The required prototype surveys vary widely with the characteristics of the estuary or coastal region and the problems to be investigated. Data required on most estuary and coastal models include hydrographic and topographic surveys, tidal elevations, current velocities (magnitude and direction), salinities, freshwater inflows, and shoaling rates and patterns. In addition, data on wave climate and dye and heat dispersion are often required.

18. To ensure that the model results are valid over the range of tidal and freshwater inflow conditions that are normally expected to occur, hydraulic and salinity field surveys are necessary for various tidal conditions and freshwater inflows. Typically, two or three such surveys are required. For example, if there is a wide variation of tidal range, surveys can be made for neap and spring tides with normal freshwater inflow. If tidal variations are small, surveys are made for various inflow conditions without regard to tidal range. A long-term salinity survey may also be useful where salinities are observed periodically at a limited number of locations to determine seasonal fluctuations.

Hydrographic surveys

19. In order for the model to be an accurate geometric replication of the prototype, detailed hydrographic surveys are required of the entire area to be included in the model. National Ocean Survey boat sheets may be used for this purpose; however, these sheets have limited value because they frequently do not show recent changes. Project (condition) surveys by the US Army Corps of Engineers are usually current but are generally limited to the immediate vicinity of a Corps project, such as a navigation channel. If the available surveys are several years old and there is doubt as to their accuracy, cross-channel profiles should be obtained at about 0.25-mile intervals to verify old surveys. In areas where the bed is subject to rapid change, hydrographic surveys should be scheduled to essentially coincide with the velocity and salinity surveys. The hydrographic surveys must include the intertidal zone between mean low water and mean high water (MHW). Recent aerial photos, especially of the bank line, are helpful in confirming the location of structures along the shore.

Topographic surveys

20. Topographic surveys are required to determine the overbank slopes immediately adjacent to the MHW line. These surveys should extend to about 10 ft above MHW. If the model is used for hurricane-surge protection studies, the topographic surveys must cover all areas that may be subject to inundation during a surge for existing or proposed conditions. In a large estuary model (e.g. Chesapeake Bay or Delaware Bay), the topography can be obtained from US Geological Survey quadrangle sheets and supplemented with field surveys as required.

Tidal observations

21. Tide records must be obtained along the length of the estuary and major tributaries. Depending on the complexity of the system, tide records should usually be obtained at intervals of 10 to 20 percent of the length included in the model. If possible, a gage should be located in the ocean, even if the datum of the gage cannot be accurately established. The data from this gage are valuable in determining whether a significant choking of the tide occurs through the estuary mouth. The datum of each gage should be determined to an accuracy of ± 0.1 ft, and all the gages should be referenced to a common horizontal datum, such as mean sea level. The gages should be put into operation about 3 months before the velocity and salinity surveys and

should be operated continuously throughout the surveys and for an additional 2 months.

Current velocity and direction

22. Velocity metering stations must be established on several ranges across the estuary and major tributaries. These current ranges should be spaced (as for the tidal stations) at intervals of 10 to 20 percent of the length of the modeled part of the estuary or coastal region. It is usually unnecessary to obtain velocity data upstream from the problem area since that part of the model can usually be adjusted using only tidal data. If the problem area is confined strictly to the entrance area, three or four velocity ranges may suffice. Depending on the width and shape of the cross-section area of the estuary at each range, one to five velocity stations should be located on each range. As many as 11 stations on a single range have been required on ranges across wide, deep estuaries such as Chesapeake Bay. For channels in the upstream reaches of the estuaries or tributaries, a single station may be sufficient.

23. Velocity observations should be made at various depths on each station. In depths of 6 ft or less, only the middepth observations are required; in depths of about 6 to 15 ft, only surface and bottom observations are normally required. In greater depths, the vertical observation interval depends largely on the expected degree of salinity stratification. In well-mixed estuaries, surface, middepth, and bottom observations are sufficient; in estuaries with a higher degree of stratification, observations should be made at the surface and bottom and at the 1/4-depth points or at depth intervals of 6 to 10 ft. Surface measurements should be made at 1 to 3 ft below the water surface, and bottom measurements at 2 to 4 ft above the bottom.

24. Current velocities (magnitude and direction) should be observed at each designated depth at each station at 0.5-hr intervals over a complete tidal cycle. Where the tides are mixed or diurnal (Pacific and gulf coasts), a complete tidal cycle refers to a 24.84-hr period; where the tides are semi-diurnal (Atlantic coast), a complete tidal cycle is a 12.42-hr period. If the number of stations is relatively small, it is recommended that sufficient personnel and equipment be assembled to monitor all stations during a single tidal cycle. If this is not possible, the survey should be conducted during as few consecutive days as possible and during a period when successive tides are predicted to have a reasonably uniform amplitude. In such cases, one

station should be established as the control station, and it should be monitored on each day of the survey period to determine the effects of varying tidal conditions on the magnitude and phasing of velocities. To minimize the time required to complete the survey, a single boat can concurrently monitor more than one station--if the stations are located close enough for the boat to monitor each station every 30 to 45 min.

Salinity

25. Salinities should be measured concurrently with velocity measurements at all ranges, stations, and depths specified for velocity observations; however, it may be necessary to extend the salinity survey upstream to the extent of saltwater intrusion. These data are sufficient to define the lateral and vertical salinity distributions throughout the system for particular freshwater inflow conditions and to evaluate the change of the salinity with tidal phase in all critical areas. However, if the velocity-measuring program does not cover a significant part of the year to adequately evaluate the response of the salinity regimen to major changes in freshwater inflow, a supplemental long-term program of salinity measurements may be required.

26. This can be accomplished by establishing a network of key salinity stations throughout the system, and salinity observations can be made at intervals (e.g., at the time of high-water slack every few days) over a period during which freshwater inflow varies from minimum to maximum. The sampling network for the long-term salinity survey can be random, in which case the variation of salinity with time is determined at each point; or, the stations can be located along the length of the main channel, in which case the longitudinal salinity profile for each sampling period is also determined. Both types of measurements have been successful, and their results are adequate to evaluate the long-term response of the salinity regimen to variations in freshwater inflow.

27. Salinity can be determined either by laboratory analysis (titration or conductivity) of samples taken from the estuary, or by in situ measurements with conductivity or inductance meters. In the latter case, some physical samples must also be obtained to ensure that the meter calibration remains stable throughout the survey period.

Freshwater inflow

28. Freshwater inflows (mean daily flows) from all tributaries to the estuary must be determined for about 2 weeks before and during the velocity

and salinity survey. Inflow data are also required during long-term salinity observation programs, but in this case mean weekly flows may be satisfactory, depending on the inflow and the estuary volume. During low freshwater discharge periods, very small individual discharges (e.g., industrial discharges of well water) may become a significant part of the total freshwater inflow and should be monitored.

Dye dispersion

29. Although field dye-dispersion tests have not generally been used for model verification, the tests should be done if dye-dispersion tests are performed in the model. A fluorescent dye should be released continuously over a 2-week period or, preferably, until a stable dye regimen is established throughout the estuary. Thus, it should be made during a period of relatively uniform freshwater inflow and may require a continuous release for 6 weeks or more. Data analysis will be complicated by dye decay, etc., during such a long period. The dye should be released at a location about two-thirds the distance from the entrance to the upper limits included in the model. Dye concentrations should be determined at surface and bottom at numerous stations throughout the estuary. The velocity and salinity station locations may be satisfactory, although additional stations along the channel center line may be desirable. The concentrations should be determined at the times of local high- and low-water slack at daily intervals during the period of rapid dye buildup, but the sampling frequency can be reduced to intervals of 3 to 7 days during the latter stages of the test.

Heat dispersion

30. If tests are made of the heat dispersion from an existing power plant, water temperatures should be monitored in the field for use in model verification. Surface temperatures should be measured on several ranges across the plume at about 1,000-ft intervals both upstream and downstream from the discharge point, and vertical temperature profiles should be obtained at several stations in the survey area. The survey coverage should be sufficient to identify the limits of the 0.6°C (1°F) temperature rise contour. At least one station should be located outside the thermal plume upstream from the discharge point and one station downstream to define the ambient water temperature. In designing the layout of the field survey stations, it is helpful to first obtain infrared aerial photos of the area to determine the size and shape of the thermal plume. Similar aerial photos should be taken

during the actual survey to obtain a better synoptic view of the thermal patterns than can be obtained with contact measurements.

Sedimentation.

31. Many model shoaling investigations for existing navigation channel projects are conducted with fixed-bed models. In this case, available periodic hydrographic surveys for a channel by the Corps of Engineers or other responsible agency will probably be sufficient for use in the model study. Channel surveys for several representative years (at least two, but preferably three or more surveys) should be analyzed to determine the distribution of shoaling throughout the length of the channel. The channel should be subdivided into several sections (usually longitudinal), and the volumes of shoaling between dredging operations should be determined for each section for each year or for each dredging season. This information is determined from the postdredging survey for 1 year and from the predredging survey, usually performed the following year. In this manner, the average percentile distribution of shoaling along the channel can be determined.

32. If shoaling tests are required over the entire width of the estuary or if no navigation channel exists, hydrographic surveys over a much broader area are required. Again, surveys are required for a period of several years and should be of sufficient detail and accuracy to develop scour-and-fill maps for the area to be studied.

Prototype data requirements

33. Perhaps the most important aspect of the design phase of a movable-bed model study is to ensure the adequacy of the prototype data. The model is constructed to conform to prototype surveys; adjustment of the model to reproduce prototype hydraulics or sedimentation patterns to an acceptable degree of accuracy is based on prototype measurements. Any errors or insufficiencies in prototype information will result in inadequate and incorrect performance of the model.

34. Prototype information required for a coastal movable-bed model study includes inlet geometry; bar configuration; adjacent beach configuration; bay geometry; wave measurements; littoral drift estimates; water surface time-histories; sediment properties of the ocean, inlet, bay, and adjacent beaches; and concurrent tidal currents in the ocean, inlet, and bay. Wind observations are also required to determine the resulting wind-driven setup or setdown of the water surface. If evaporation or precipitation appears to be

important or if freshwater inflows constitute a significant part of the flow in the region, these should also be observed. The occurrence of storms of low-return frequency should be noted in the history of the study area since large volumes of sediment can be displaced during these storms. Hydrographic and wave observations should also be made frequently enough to detect seasonal and yearly fluctuations.

35. A longer data-collection period is needed for a movable-bed study than for a fixed-bed model. The period length also varies with the data type; i.e., longer term wave data are needed than tide level and current data to calibrate a movable-bed model. Prototype observations for several consecutive years before the model study will allow an evaluation of both short- and long-term tendencies of the region and the selection of a typical period on which to base the model verification. If these prototype data are not available, a program to collect sufficient information may have to be initiated before beginning the model study.

Verification

Fixed-bed models

36. Verification of a fixed-bed model is generally accomplished in three phases: (a) hydraulic verification, which ensures that the tidal elevations and times and the current velocities and directions are in proper agreement with the prototype; (b) salinity verification, which ensures that salinity phenomena in the model correspond to those of the prototype for similar conditions of tide, ocean salinity, and freshwater inflow; and (c) fixed-bed shoaling verification, which ensures acceptable reproduction of prototype shoaling distribution. In addition, dye-dispersion verification is accomplished if the results of a prototype dye-tracer study are available.

37. Since discrepancies between model and prototype observations are likely, the effects of various plans tested in the model on the basis of direct model-to-prototype comparisons cannot be evaluated. After the model has been verified, a series of observations are made throughout the model to define existing or base conditions in the model. The plan test results are then evaluated on the basis of model-to-model comparisons to determine the changes caused by the plan.

38. The basic objective of the fixed-bed model shoaling verification is to identify a model sediment that will move and deposit under the influence of model forces in the same manner that the natural sediment moves and deposits under the influence of natural forces. Because no satisfactory similitude laws have been developed for estuarine and coastal sedimentation, the development of the modeling shoaling test procedure is more an art than a science at this time. The appropriate time and volume scales for the shoaling tests must be determined by trial and error.

39. Many variables are involved in identifying a suitable operating technique for use in the model, and each must be resolved by trial and error in the model. The most significant variables include: (a) shape, size, gradation, and specific gravity of the model sediment; (b) method, location, duration, and quantity of model sediment injection; (c) rate of freshwater inflow; (d) magnitude of tide; (e) height, direction, and period of ocean waves; (f) length of model operation; and (g) readjustment of model roughness. The model water temperature must be closely monitored since similar shoaling tests run with different water temperatures often give significantly different results.

40. In general, finely ground gilsonite is used to simulate suspended sediment, and granulated plastic, nylon, or other similar material is used to simulate bed-load sediment. Gilsonite is an asphaltic base material with a specific gravity of about 1.03; it is usually graded to pass a Tyler No. 24 screen (0.8 mm) and is retained on a Tyler No. 35 screen (0.4 mm). No attempt is made to model all characteristics (e.g., fall velocity) of a particular suspended sediment. Commonly used granulated materials (with specific gravities) include polystyrene (1.03 to 1.09), nylon (1.13 to 1.15), Tenite Butyrate (1.18 to 1.20), coal (1.4), and naturalite (1.7). These materials are available in various regular shapes such as cubes and cylinders or in irregular crushed shapes. The selected material is usually injected into the model in slurry form as either a point or line source, although it is occasionally spread over the entire problem area before starting the model. After completing the injection of the shoal material, the model is normally operated for several tidal cycles to allow enough time for the material to be dispersed by the currents. Waves should only be generated when the shoaling problem area is in the estuary or harbor entrance area and thus subject to ocean wave action. Because of the distorted model scales, waves in the model that

represent prototype conditions cannot be reproduced; rather, the model waves are adjusted to simulate the degree of agitation required so the model sediment can be moved and deposited by the tidal currents.

41. Because the shoaling test technique is developed by trial and error, the validity of the shoaling verification is highly dependent on the quality and quantity of the available prototype data. Surveys of the problem area should be available for a period of at least 2 and preferably 3 or more years in order that average annual conditions can be determined. The problem area is subdivided into sections, and the average annual prototype shoaling rate is determined for each section. These rates are then converted to percentages of the shoaling rate for the entire problem area, and this is the percentage distribution that is reproduced in the model. When an acceptable reproduction of the distribution pattern has been achieved, the volume of material recovered from the problem area in the model can be equated to the prototype shoaling rate to establish an approximate shoaling volume scale. The duration of the model test can also be equated to the prototype period for which the shoaling rate was developed to determine the shoaling test time scale.

42. Since all conditions cannot be duplicated between model and prototype, a precise duplication of the shoaling distribution pattern cannot be expected; e.g., the effects of overdepth or advance maintenance dredging may be difficult to simulate in the model unless the dredging practice is consistent from year to year. Because the models are fixed-bed, the effects of local scour or nearby deposition, and the changes in cross section resulting from scour or unusual dredging, cannot be simulated. At the termination of a model shoaling test, all material in motion deposits immediately in place, resulting in some model shoaling in reaches that experience none in the prototype. Disposal methods (e.g., instantaneous dump) are normally not reproduced in the model. If there is a return of material to the channel from the disposal area, this source of material may not be reproduced in the model shoaling tests.

43. If prototype data are not available for a detailed shoaling verification, any model shoaling tests are of a qualitative, rather than quantitative, nature. The relative shoaling tendencies of the various plans tested can be compared, but shoaling rates cannot be predicted. In this case, the only verification possible is the intuitive judgment of the model operator as

to whether or not the model shoaling pattern looks reasonable. The development of more than one shoaling test technique may be necessary to simulate the effects of various sediment sources.

Movable-bed models

44. Verification of a movable-bed model is, theoretically, more difficult than for a fixed-bed model. The purpose of a movable-bed model is to simulate the evolution of the coastal features. This evolution takes place in response to many factors, but primarily to the sediment washed from adjacent beaches by wave action, to erosion of the inlet channels by tidal currents, and to entrapment and resuspension of material at the bars and disposal sites of the ocean and bay systems. These same factors must be included in the model to simulate degree as well as type of bathymetry evolution. Since a movable-bed model simulates shoaling and scouring patterns, the requirement that the model also simulate the basic hydraulic quantities (tidal heights, tidal phases, velocities, etc.) is somewhat relaxed.

45. In practice, and contrary to the above discussion, the verification of a movable-bed model is a little easier than for a fixed-bed model since the experimenter has more variables available with which to work to achieve the desired verification. The validity of tests of proposed improvement plans in a movable-bed model is based on the following premise: if model reproduction of the prototype forces known to affect movement and deposition of sediment (tides, tidal currents, waves, etc.) produces changes in model bed configuration similar to those observed in the prototype under similar conditions, the effects of a proposed improvement plan on the movement and deposition of sediment will be substantially the same in both model and prototype.

46. Trends and magnitudes of prototype bed movement under existing conditions are determined primarily through detailed comparison of two or more periodic prototype surveys of the area under study. The time between the earliest and latest surveys used in this comparison becomes the verification period, and the movable-bed part of the model is molded to conform to the prototype survey at the beginning of the verification period. The model is then operated under conditions that existed in the prototype during the verification period until model bed configurations throughout the problem area are in conformance with those shown by the prototype survey at the end of the period. Model bed movement is then considered to be verified, or in proper adjustment, if changes in the model bed configurations during the verification

period agree reasonably well with those that occurred in the prototype during the corresponding period. However, when a model is operated in this fashion, basic similitude rules are relaxed, and the model does not reproduce prototype forces. Only the sediment motion at a particular point during a particular time period is being modeled. Any major change in hydrography may introduce errors into the results.

47. One very important reason for the verification of a movable-bed model is the establishment of the time scale with respect to bed movement. The model-to-prototype time scale for bed movement cannot be computed from the linear scale relations because the interrelation of the various prototype forces affecting movement and deposition of sediment is too complicated for accurate definition and, consequently, is much too intricate to permit establishment of mathematical scale relations for each component of force. The model-to-prototype time scale for bed movement is therefore determined empirically during the model verification; i.e., the actual time required for the model to reproduce certain changes that occurred in a given period of time in the prototype is used to determine the model time scale for bed movement.

Time and Cost Estimates

48. A generalization of the time and cost requirements for physical models is difficult because of the wide variety of model sizes, problems to be investigated, and amount of detail required from test results. Models generally vary in size from about 5,000 to 60,000 sq ft excluding extremes such as the Gastineau Channel model (1,600 sq ft) and the Chesapeake Bay model (340,000 sq ft). The cost of model design and construction varies not only with the area of the model but also with the complexity of the model geometry (e.g., pier slips and braided channels, as opposed to relatively flat bay bottoms and straight channels).

49. Design and construction costs by the US Army Engineer Waterways Experiment Station (WES) have been on the order of \$12 to \$15 per square foot (March 1976); however, this does not normally include the (direct) cost of major appurtenances (such as tide generators, wave generators, hurricane surge generators, and water supply pumps), the model shelter, water supply sump, or model instrumentation. These items are obtained with plant funds, and WES is reimbursed by indirect charges to all projects. Construction time also varies

with both the size and complexity of the model (generally 1 to 8 months). Most of the model design is accomplished before initiating model construction; however, the design effort is usually continued into the early stages of construction. The design is usually initiated 1 to 3 months before construction. If prototype hydraulic, salinity, and shoaling data are available, they can be analyzed during this period.

50. After completion of the model construction, a substantial period of time is required for model verification. Since this is a trial-and-error process, it is difficult to predict the time and cost required even for a specific case. The time required for hydraulic and salinity verification varies from about 3 to 15 months, depending on the size of the model, the complexity of the estuary, the amount of prototype data available, the number of conditions (combinations of tide and freshwater discharge) to be reproduced, the skill of the model personnel, and "luck." Operating costs during this period will vary from about \$8,000 to \$15,000 per month (March 1976) depending on the number of operating personnel, the amount of support (shops, molding, photography, drafting, etc.), and the amount of materials (especially salt) required. Fixed-bed shoaling verification can also be a lengthy process requiring from 2 weeks to 3 months for each reach to be studied. Monthly costs for shoaling verification will often be 10 to 20 percent less than during hydraulic and salinity verifications because fewer operating personnel are required.

51. The testing program for a single study will generally require from 1 month to 1 year. However, several studies will probably be conducted during the life of any one model. Operating costs will average between \$10,000 and \$15,000 per month (March 1976). Costs may be significantly higher during extensive dispersion studies because of the greater personnel and data-reduction requirements.

52. After completion of a model study, the results are either published in a single comprehensive report or in a series of reports on specific studies conducted in the model. Test results are furnished to the sponsor in preliminary form as soon as they are available. Preparation and publication of a final report usually requires about 9 months at a cost of approximately \$10,000 (March 1976).

APPENDIX C: NUMERICAL MODELS

Uses of Numerical Models

1. Numerical models use computational methods to solve mathematical expressions describing physical phenomena. Computational methods such as approximation and iteration performed by high-speed digital computers allow solution of complex equations that cannot be solved by analytical methods.

2. Numerical models used in hydraulics problems are of two principal types--finite difference and finite element. Numerical models are also classified by the number of spatial dimensions over which variables are permitted to change. Thus, in a one-dimensional (1-D) flow model, currents are averaged over two dimensions (usually width and depth) and vary in only one direction (usually longitudinally). Two-dimensional (2-D) models average variables over one spatial dimension, either over depth (a horizontal model) or with width (a vertical model).

3. Numerical models are used for the same types of problems as physical models (see Appendix B) except that numerical models are considered to be better for storm surge studies and most sedimentation studies. Flows where density currents play a significant role are well reproduced in physical models and appear to be so in three-dimensional (3-D) numerical models, although 3-D numerical models are relatively new, and experience with them is limited.

4. Numerical modeling provides much more detailed results than analytical methods and may be substantially more accurate, but it does so at the expense of time and money. However, once a numerical model has been formulated and verified for a given area, it can quickly provide results for different conditions. In addition, numerical models are capable of simulating some processes that cannot be handled in any other way. They are limited by the modeler's ability to derive and accurately solve mathematical expressions that truly represent the physical processes being modeled. The skill of a modeler in applying a particular model is also important.

5. Overall, numerical model studies are less expensive to conduct than physical models and do not share the difficulties of physical models with regard to the scaling for sediment studies. Although much development and

experience are needed, numerical models provide the greatest hope for accurately modeling the resuspension and transport of disposal mound material.

6. The use of numerical models for dredged material aquatic disposal sites (DMADS) can be classified into two groupings: (a) models for predicting the short-term behavior of disposal material during and immediately following disposal; and (b) models for predicting the long-term fate of the mound material in terms of resuspension, transport, and redeposition. The former models have been developed and used since the early 1970s. Although there are models that have the potential for use in the latter class, some further development and much application and confirmation are needed. A numerical model study of the second kind had not been conducted for DMADS at the time of this writing.

Available Numerical Models

7. Selection of a computer program to use in a numerical modeling study depends on the processes to be modeled and the skill and experience of the modeler. The program selected should use equations that adequately describe the transport processes of interest without computational overkill. The best choice is often the program that the modeler is most familiar with.

8. Selection of a 1-D, 2-D, or 3-D modeling program is determined by the processes in the area to be modeled. Many inland waterways can be adequately modeled by 1-D steady-flow programs. Most tidal waterways require unsteady flow computations and at least a 2-D program to describe transport processes. Some require a 3-D program, particularly broad estuaries with some density stratification and coastal regions.

9. The prototype is described by a digital model that gives the locations of the boundaries and imposes a computational mesh over the area. At each location in the mesh, the geometry (cross-sectional area, depth, or width) and physical characteristics (boundary roughness, sediment characteristics, etc.) are described. The discretization must use enough points to adequately describe the geometry and to represent the scale of flows and transport processes. For example, a 2-D model of a tidal waterway with two deep channels separated by shallows must have a minimum of five points in the cross section, and more are preferred.

10. Time steps must be chosen to at least describe the processes involved. For example, adequate resolution of a 12.4-hr tide will require that time steps be less than 2 hr long (tidally averaged computations are an exception). Transport processes, particularly sediment transport, will require smaller time steps, usually less than 30 min. Some computational schemes, such as explicit time-stepping or alternating direction implicit, will impose smaller time steps in order to obtain a stable and accurate solution.

11. Within the Corps of Engineers, 1-D, 2-D, and 3-D numerical models are available. The following paragraphs describe those that the Corps uses on sedimentation problems. Other models are available from outside the Corps.

1-D models

12. Several 1-D sediment transport models are available, with HEC-6 (US Army Engineer Hydrologic Engineering Center 1977*) being the most widely used. HEC-6 computes both flow and sediment transport. It is designed to analyze scour and deposition in rivers and reservoirs. HEC-6 and similar models are used when the flow is unidirectional and constrained to follow well-defined channels. It calculates transport of sand, silt, and clay.

2-D models

13. Two-dimensional numerical models include those that are integrated over depth (horizontal models) and those that are integrated over width (vertical models).

14. Two-dimensional, depth-averaged models have been most commonly employed in the investigation of tidal flows in inlets, bays, and estuaries. Two distinctly different formulations have been employed: finite difference and finite element. Both types are being used at the US Army Engineer Waterways Experiment Station (WES).

15. The TABS-2 system is a complete system built around finite element hydrodynamic and transport models. It provides for digitizing maps, automatic generation of computational meshes, modeling flow and transport, and analysis and display of results. A variety of graphical displays permit rapid plotting of model results to assist the modeler in understanding the results. Graphical output includes contour and factor maps of sediment concentration and deposition and erosion, vector plots of sediment transport rates, and

* See References at the end of the main text.

tracer path plots. TABS-2 is designed for use by engineers who are not computer experts. An interactive English-language interface sets up computer runs and prompts the user for needed information. The system is in widespread use by the Corps and has been applied to more than two dozen projects.

16. Horizontal 2-D modeling of sediment transport is performed with TABS-2 by STUDH (Thomas and McAnally 1985). It is designed for situations where flow and transport can be satisfactorily described by depth-integrated equations. STUDH computes the transport of sand, silt, or clay. It obtains flows either by specification or from the TABS-2 flow model, RMA-2V. It calculates transport due either to currents alone or to currents plus short-period waves (nonbreaking). The program allows for wetting and drying of tidal flats and consolidation of fine sediment with overburden and time.

17. The model WIFM (WES Implicit Flooding Model) is a 2-D, depth-integrated finite difference code that is frequently used by WES. WIFM evolved from early work by Leendertse (1967, 1970). The model and its application have been refined and significantly improved at WES and have been described at different stages of development by Butler (1978). WIFM is a general long-wave model that can be used for simulation of tides, storm surges, tsunamis, etc. It allows flooding and drying of land cells near the shoreline. It is used to determine tidal elevations and velocities in the two horizontal coordinate directions.

18. The 2-D, depth-integrated model, WIFM-SAL (Schmalz 1985), is a transport companion model for WIFM. Presently only one conservative solute is included for the transport, but the model is being modified to include sediment transport.

19. Coastal processes of tides, waves, wave-induced currents, and sediment transport can be modeled by using the 2-D numerical modeling system CIP (Coastal and Inlet Processes). The system utilizes: WIFM for tides; the Regional Coastal Processes Wave Propagation model (RCPWAVE) for waves; the model CURRENT for wave-induced currents; and a sediment-transport model for transport of sediments due to the combined action of tides, waves, and wave-induced currents. All four models generally use the same computational grid for a given set of conditions.

20. WIFM determines tidal elevations and velocities in two horizontal coordinate directions. RCPWAVE is a linear short-wave model that considers

the transformation of surface gravity waves in shallow water, including the processes of shoaling, refraction, and diffraction due to bathymetry, and allows for wave breaking and decay within the surf zone (the region shoreward of the breaker line). Information from RCPWAVE can be used directly as input to the wave-induced current and sediment transport models. CURRENT computes the wave-induced currents that result when waves break and decay in the surf zone. In general, such breaking induces currents in the longshore and cross-shore directions, with resulting changes in the mean water level. These currents play a major role in the movement of sediment in the nearshore region.

21. The sediment-transport model predicts the transport, deposition, and erosion of sediment in open coastal areas as well as in the vicinity of tidal inlets. It accounts for both tides and wave action by using for input the results of WIFM, RCPWAVE, and CURRENT in terms of tidal elevations and currents, wave climate information, and wave-induced currents and setups at the centers of grid cells. The model computes transport separately for straight open coastal areas and for areas in the vicinity of tidal inlets. In the case of a straight open coastal area, transports inside and outside the surf zone are treated separately. Inside the surf zone, it is the wave-breaking process that is primarily responsible for the transport of sediment. Beyond the surf zone, waves are not breaking, but currents (tidal, littoral, rip, etc.) still transport sediment although the sediment load is much smaller than in the surf zone.

22. Laterally averaged models are applicable in studies of relatively deep narrow channels with small radius of curvature in which lateral secondary currents of appreciable magnitude do not develop. Since fewer estuarine/coastal systems meet this criterion, work on models of this type has been more limited than on the depth-averaged models. However, work performed during the last few years has produced a useful model, LAEM (Laterally Averaged Estuarine Model), by Edinger and Buchak (1981). This model has been used to investigate the effect of navigation channel deepening on salinity intrusion in the lower Mississippi River (Johnson, in preparation). Additionally, this model has been recently modified to include sediment transport (Johnson, Boyd, and Keulegan in preparation).

3-D models

23. Two sets of 3-D numerical models are in use by the WES--the RMA series and CELC3D (Coastal, Estuarine, and Lake Currents). These models could possibly be used for assessing the behavior of the disposal mound and fate of resuspended sediment. Both models have a free surface, are time-dependent, and allow for stratification and complex geometry.

24. The RMA series of programs models flow and transport in three dimensions using the finite element method. Program RMA-8 computes water levels and currents for constant density flows. RMA-10 computes water levels, currents, and salinity/temperature transport for flows with density gradients. SEDIMENT 8 computes sediment transport for sand, silt, or clay.

25. The RMA programs were designed for use in situations where transport of sediments and other materials is important. They permit parts of the computational mesh to be 2-D while the problem area is modeled in three dimensions, thus ensuring economical operation.

26. The 3-D finite difference model CELC3D (Sheng 1983, 1984) simulates hydrodynamics and transport for temperature, salinity, and sediment. Special features of CELC3D include: (a) a "mode-splitting" procedure that allows efficient computation of the vertical flow structures (internal mode); (b) an efficient alternating direction implicit (ADI) scheme for the computation of the vertically integrated variables (external mode); (c) an implicit scheme for the vertical diffusion terms; (d) a vertically and horizontally stretched coordinate system; and (e) a turbulence parameterization that requires relatively little tuning. Sheng (1983) has made numerous test applications of the model that demonstrate well the model's utility and accuracy.

27. For a scenario of repeatable hydrodynamics, CELC3D can be combined with the sediment-transport algorithm for long-term computations on the order of weeks, months, or longer. CELC3D provides for the resuspension, transport, and deposition of coastal sediment where sediment particle dynamics are modeled by a consideration of particle groups and coagulation processes. Detailed dynamics within a turbulent boundary layer, under pure wave or wave-current interaction, are evaluated by means of a turbulent transport model.

Models for short-term fate

28. In order to assess the physical capacity of an aquatic disposal site, a knowledge of the short-term fate of dredged material from individual disposal operations is required. Questions to be answered before addressing

the long-term transport of material from a site to determine its capacity are how much material is initially deposited in the site, in what form, and how large the area is over which the deposition occurs. The disposal site environment, the composition of the disposed material, and the method of disposal are the major factors in determining answers to these questions.

29. Three mathematical models of the physical processes determining the short-term fate of the dredged material have been developed for use by the Corps: DIFID (Disposal from an Instantaneous Dump), DIFCD (Disposal from a Continuous Discharge); and DIFHD (Disposal from a Hopper Dredge). The discussion below relates to all three models. For more details, the reader should refer to Johnson (in preparation).

30. In each model, the behavior of the material is assumed to be separated into three phases: convective descent, during which the dump cloud or discharge jet falls under the influence of gravity; dynamic collapse, occurring when the descending cloud or jet either impacts the bottom or arrives at the level of neutral buoyancy at which descent is retarded and horizontal spreading dominates; and long-term passive dispersion, commencing when the material transport and spreading are determined more by ambient currents and turbulence than by dynamics of the disposal operation. These computer programs enable the computation of the physical fate of dredged material disposed in open water.

31. DIFID models an instantaneous dump. If all the material leaves the disposal source within a few seconds, the assumption of an instantaneous dump is adequate. DIFCD models a continuous discharge. Pipeline disposal operations or perhaps hopper dredge disposal from a single bin where either the speed of the vessel or the ambient current is strong enough to result in severe bending of the convective descent jet should be modeled with DIFCD. If disposal is from essentially a vertical discharge from a basically stationary source, DIFHD should be applied. With this model, the continuous nature of the discharge is allowed while retaining the bottom collapse feature of DIFID that gives a radial spread on the bottom.

Advantages and Disadvantages of Numerical Models

32. Occasionally, a decision must be made whether it would be more appropriate to develop a numerical modeling capability or construct a physical

hydraulic model to investigate certain physical phenomena under consideration. Each type of model (numerical or physical) has certain inherent advantages, but each type also possesses disadvantages with respect to operational capabilities and cost-effectiveness. These characteristics must all be weighed by the sponsoring organization to decide which type of modeling effort is most appropriate for the existing situation.

33. Advantages of numerical modeling that should be considered are:

- a. The development is usually a one-time event. Numerical models are usually formulated for general situations and are applied to specific localities. Hence, the same numerical model can be used at many different prototype locations by selecting the appropriate grid generation layout and by applying correct boundary conditions.
- b. The bottom topography can be changed rapidly as the model study progresses from an existing condition situation to evaluation of various plans for improvement.
- c. Structures and other boundary conditions can be altered rapidly to accommodate changed conditions.
- d. Input parameters can be adjusted as prototype conditions indicate (e.g., riverflow can be incremented; wave heights, periods, and directions can be varied; and wave spectra can be introduced to the study).
- e. Sensitivity analyses of specific variables can be performed rapidly to optimize certain input conditions.
- f. Operational costs are probably less than many hydraulic model studies, as a numerical model does not require the large expanse of space needed for physical models or the support equipment needed for most hydraulic models (e.g., wave and tide generators).
- g. Physical processes can be simulated that a physical model is not ordinarily capable of reproducing (storm surge, sedimentation, Coriolis, wind stress, wind-wave generation over large areas, simultaneous refraction and diffraction, etc.).
- h. Iterative solutions to complex problems that cannot be solved by analytical methods are available.
- i. Scale effects are precluded in a numerical model (Reynolds and Froude similarity criteria can both be satisfied simultaneously).
- j. Minimal equipment is usually necessary at the local level. Computer terminals with telephonic communication capability to a mainframe machine is all that is usually required. Some numerical models are capable of operating on microcomputers.

34. Disadvantages of numerical modeling are:

- a. A fixed numerical grid implies fixed measurement or computa-

tional points (usually at the center of grid cells), whereas measurement and data stations can be placed anywhere in a physical model.

- b. It is not always possible to mathematically describe the physical processes involved, as the interactions between two or more variables may result in complex phenomena not entirely understood.
- c. Numerical models may incur stability problems (e.g., nonclosure of turbulence models at practical scales).
- d. Difficulties may arise in fitting real-world boundaries with numerical computational grids, although development of boundary-fitted coordinate schemes, which will reduce this problem, is under way.
- e. Some highly nonlinear phenomena, such as breaking waves in the surf zone or stability testing of rubble-mound structures, do not lend themselves to numerical modeling.
- f. Boundary conditions may be difficult to simulate (e.g., power plant intakes, spillways, and locks).
- g. The success of a numerical modeling endeavor depends on the ability of the numerical modeler to accurately apply the model that represents the physical processes.

Field Data Requirements*

35. Field data requirements for numerical models are essentially the same as for physical models (see Appendix B) except for sediment data. Sediment data requirements for numerical modeling include details of grain-size distributions, suspended sediment concentrations, and characteristics of the bed.

Grain sizes and settling velocities

36. Sediment grain sizes and settling velocities must be analyzed before determining values to be used as model input. The input data needed by some models are effective grain size and settling velocity. This is particularly important for settling velocity since measurements are made in quiescent water (settling tubes) whereas the effective settling velocity includes turbulent resuspension in the flow.

37. Cohesive sediment exhibits different settling velocities in the laboratory than in the natural environment. This is caused by changes in

* Paragraphs C35 through C48 were taken from Appendix K of Thomas and McAnally (1985).

aggregation of the particles during sampling and storage. Laboratory measurements of settling velocities should usually be supplemented by onsite measurements with a Horizontal Niskin Sampler. See Fisackerly et al. (in preparation) for a discussion of the Horizontal Niskin Sampler measurements.

Concentrations

38. Sediment concentration profiles in the vertical are needed even in a 2-D model study. The concentration profiles can be used to deduce settling velocities and critical shear stresses in some cases.

39. Under conditions of high suspended sediment concentrations and steep concentration gradients (typical in estuaries, sometimes in reservoirs), discrete sampling of concentrations in the vertical can lead to erroneous conclusions. The behavior of very fine sediment in high concentrations leads to nonsmooth concentration profiles. If such conditions exist, density profiles should be obtained with a densimeter (Parker and Kirby 1981). Density profile information can also be used in specifying bed conditions and determining critical shear stresses for erosion.

40. Bed roughness values for the sedimentation model may be different from those used in hydrodynamic modeling. For hydrodynamic modeling, the total resistance of the water is needed, including bed (surface and form roughness) and channel geometry (meanders, etc.). For sedimentation modeling, channel geometry is not used, and the modeler may even need a separation of surface and form roughness.

Bed density, structure, and consolidation coefficients

41. Density measurements can be made on cohesive bed sediment. The profile of bed density with distance into the bed can be determined in situ by a densimeter (Parker and Kirby 1981) for low densities and by lab tests on slices of short cores for compacted sediment. Density profiles in newly deposited material may also be measured in the lab by analyzing deposits made in a settling column from resuspended bed samples. Inspection of cores and lab analyses of slices from the cores can provide information on layering of various sediments.

Critical shear stresses

42. Indirect methods. Indirect estimates of critical shear stresses for deposition and erosion of cohesive sediment can be made by measuring

certain parameters and comparing them with values obtained by lab tests on similar sediments. Measurements of mineralogy, cation exchange ratio, and sodium adsorption ratio can be used to characterize the sediment and thus the critical shear stresses. For examples, see Krone (1972, 1978) and Ariathurai and Arulanandan (1978).

43. Direct methods. Direct methods of obtaining critical shear stresses involve performing flume tests on sediment samples, then calculating the critical shear stresses from test results. These tests usually require large sample volumes, specialized flumes, or both.

44. A second direct method uses a rotating cylinder apparatus in which a core of sediment is placed. Smaller samples can be used, but the method may require remolding of samples and does not work for very loose, fresh deposits.

45. Direct methods for obtaining critical shear stresses are fairly difficult to perform, and the results may be affected drastically by seemingly minor test procedures. An improved method is needed.

Dispersion coefficients

46. Dispersion coefficients cannot be measured directly. They are a function of flow characteristics, the vertical sediment concentration profile, and the computational mesh.

Deposition/erosion patterns

47. Deposition/erosion patterns and volumes are usually obtained by comparing successive hydrographic surveys. The results are complicated by surveying errors and inaccuracies and by dredging activities. For a discussion of some of the problems involved and how to handle them, see Letter and McAnally (1977, 1981).

48. Areas of cohesive sediment beds sometimes have thick layers of very low density deposits on the bed. These deposits are often transitory, occurring during periods of low velocity and resuspending if velocities increase (such as tidal cycle behavior in coastal areas). Low density deposits can cause acoustic sounding equipment to provide depth measurements that are seriously (several feet) in error. For such conditions, densimetric surveys may be needed.

Time and Cost Estimates

49. Like physical model studies, generalizing time and cost estimates for numerical model studies is difficult because of the wide variety of problems and areas to be modeled. Development and verification of a computational mesh for a given area can range from 3 months at a cost of \$30,000, to 18 months at a cost of \$200,000. Testing subsequent to verification can be of almost any duration, from 1 month to several years.

50. After completion of a model study, the results are provided to the sponsor as soon as they are available and then published. Preparation and publication of a report usually takes 2 months to 1 year and costs \$1,500 to \$15,000.

APPENDIX D: HYBRID MODELS

1. Choosing among modeling, field tests, and analytical studies of dredged material movement requires that the engineer make balanced trade-offs among time, cost, and accuracy. Even choosing a model results in some trade-offs since each model has its own strengths and weaknesses.

2. Field (prototype) data collection and analysis serve both as important aspects of the other solution methods and as independent methods. They form an indispensable element in verification of numerical and physical models. And, to a limited extent, field data can be used to estimate the estuary's response to different conditions of tide and river discharge. Obtaining sufficient temporal and spatial data coverage in the field, however, is a formidable and difficult task. Field testing of structural alternatives costing millions of dollars is too costly and too risky.

3. Analytical solutions are those in which answers are obtained by use of mathematical expressions. Analytical models usually combine complex phenomena into coefficients that are determined empirically. The usefulness of analytical solutions declines with increasing complexity of geometry or increasing detail of results desired.

4. Numerical models employ special computation methods, such as iteration and approximation, to solve mathematical expressions. They are capable of simulating some processes that cannot be handled any other way. Numerical models provide much more detailed results than analytical methods and may be more accurate, but they do so at the expense of time and money. They are also limited by the modeler's ability to formulate and accurately solve mathematical expressions that truly represent the physical processes being modeled.

5. Physical scale models have been used for many years to solve coastal hydraulic problems. Physical models of estuaries can reproduce tides, freshwater flows, longshore currents, and three-dimensional variations in currents, salinity, and pollutant concentration. Conflicts in similitude requirements for the various phenomena usually force the modeler to neglect similitude of some phenomena in order to more accurately reproduce the more dominant processes.

6. The preceding paragraphs have described the four principal solution methods and some of their advantages and disadvantages. Common practice has been to use two or more methods jointly, with each method being applied to

that portion of a complex coastal hydraulic problem to which it is best suited. For example, field data are usually used to define the most important processes and to verify a model that predicts hydrodynamic conditions in an estuary. Combining physical modeling with numerical modeling is termed a hybrid modeling method. Combining them in a closely coupled fashion that permits feedback among the models is termed an integrated hybrid solution. By devising means to integrate several methods, the modeler can include effects of many phenomena that previously were either neglected or poorly modeled, thus improving the accuracy and detail of the results.

7. The hybrid modeling technique is exemplified by the first integrated hybrid study performed--the Columbia River entrance. A detailed description is given by McAnally et al. (1984a,b).*

8. The following paragraphs describe the operations involved in a general application of the hybrid modeling method. Figure D1 shows the general sequence of steps performed in applying the hybrid solution method as used in the Columbia River entrance studies.

- a. Step 1: Define problem. Each problem is first tested with existing conditions, and then one or more plans representing channel or structural changes is tested.
- b. Step 2: Define boundary conditions. The numerical solution methods require that the user supply proper boundary condition information. In estuary sedimentation problems, this includes freshwater inflows at the upstream boundary, water surface elevations, wave conditions, salinities, and sediment concentrations at each time step. Normally, the hydraulic data would be obtained from a physical model.
- c. Step 3: Select initial conditions. The initial conditions are defined, including the initial bathymetry of the study area.
- d. Step 4: Predict hydrodynamic and salinity conditions. Driven by the physical model current and water surface elevation measurements at selected points, the hydrodynamic model interpolates the data and calculates currents and water surface elevations at each computation point. By integrating the physical model current measurements over depth, the effect of density currents is indirectly incorporated into the two-dimensional numerical model, provided that physical model measurements are made at appropriate points. Wave conditions in the entrance are computed by a numerical wave refraction/diffraction model, and longshore currents are predicted analytically by the Longuet-Higgins technique (US Army Corps of Engineers, Coastal Engineering Research Center 1984). These

* See References at the end of the main text.

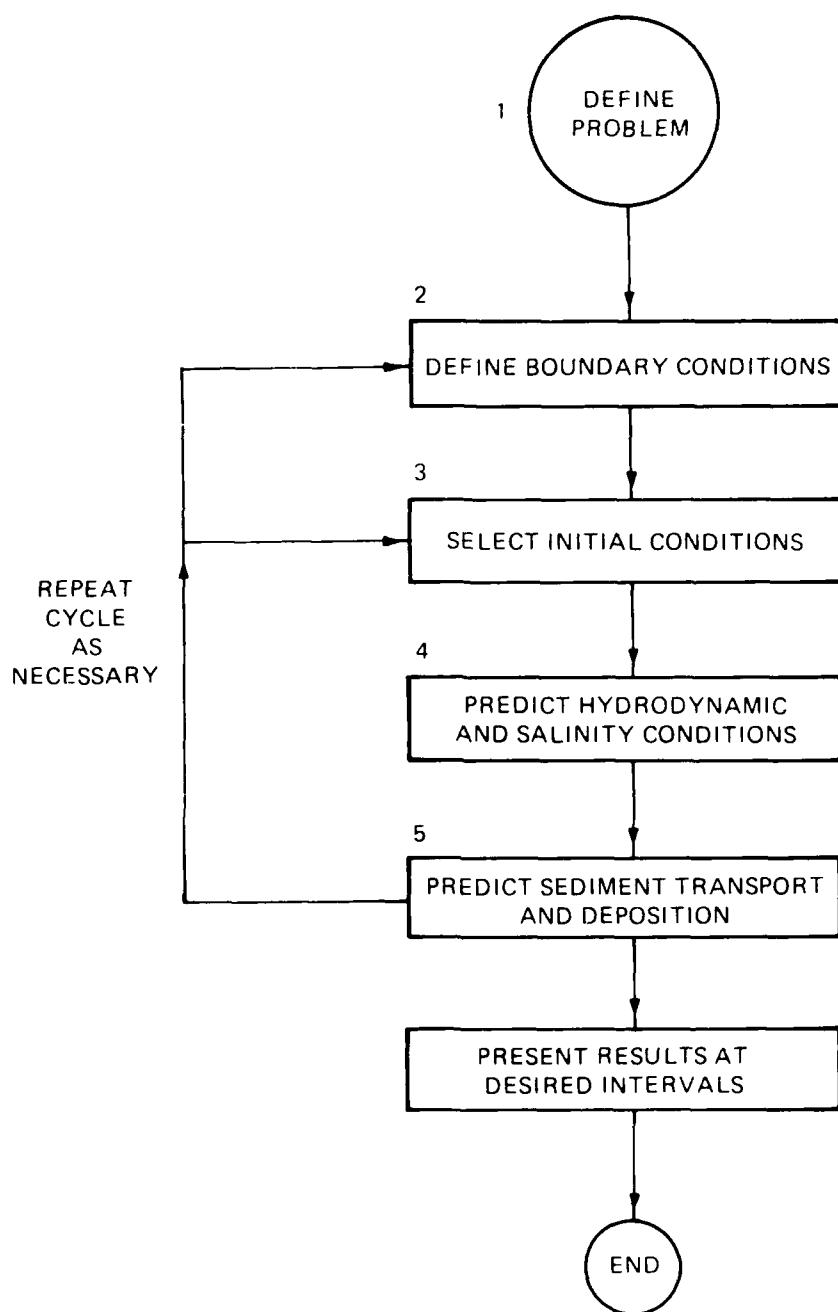


Figure D1. Steps in the hybrid solution process
(after McAnally et al. 1984a)

currents are linearly superposed on tidal currents predicted by the physical and numerical models.

- e. Step 5: Predict sediment transport and deposition. Using the hydrodynamic data from Step 4, the numerical sediment transport model predicts sediment transport, erosion, and deposition. Depth changes are monitored and computations halted when changes become large enough to change the hydrodynamic response of the system. If this occurs, the solution process returns to Step 3, and the bathymetry is updated and new hydrodynamics are computed before resuming sediment modeling.

9. When a period of modeling is complete for a given combination of hydrodynamic events (waves and currents), the solution process returns to the boundary condition step.